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A PRELIMINARY ASSESSMENT OF THE NIPIGON RIVER LANDSLIDE (1990)

Report No 91-49-K

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ABSTRACT

On April 23, 1990, a large size landslide occurred on the east bank of the Nipigon River, 8 km downstream of Alexander GS. Trow Ontario Limited, after conducting a post-failure investigation for the Ministry of Natural Resources and TransCanada Pipelines concluded that the landslide was of a retrogressive type caused by a combination of natural and man made conditions such as: rapid spring thaw, weak soils in the area, groundwater recharge from the cleared area and river level drawdown due to the Alexander GS operation. In this report, all of the available data from Trow's report and the airphoto study by Bird and Hale Limited are reviewed and a parametric slope stability analysis is performed to rank the importance of each factor that may have contributed to the landslide. It is postulated that an initial bank slope failure occurred as a result of high moisture conditions in the bank and the river drawdown effect, and then the failure retrogressed by liquefaction of a sand layer and softening of the clay layers under very high groundwater pressures. The previous layer beneath the soft clay connecting to the recharge area provided the required high groundwater pressures.

On the basis of this analysis, it is inferred that the impact of the river level fluctuations on the occurrence of the large size landslide of 1990 is relatively a minor one. The subsoil and groundwater conditions in the area behind the bank slope were the key elements in producing a large retrogressive type of landslide.

These postulations are based on the assumed soil properties and groundwater conditions. Therefore, a field checking of these parameters, and a study of the effect of river drawdown on the slope stability are recommended.

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TABLE OF CONTENTS

	Page
1.0 INTRODUCTION	1
2.0 THE SITE CONDITIONS	2
2.1 Bank Slopes and Site Drainage	3
2.2 Past Slope Failures	3
2.3 The 1990 Failure Site	4
2.3.1 Soil Stratigraphy	5
2.3.2 Geometry of the Slope	6
2.3.3 River Levels and Drawdown	6
2.3.4 Groundwater Conditions in the Bank Slope	7
3.0 THE POSTULATED FAILURE MECHANISM	8
3.1 Initial Slide	9
3.1.1 Parametric Study	9
3.1.2 Results of the Parametric Study	11
3.1.3 The Proposed Failure Mechanism of the Initial Failure	12
3.1.3.1 Conditions Prior to the Initial Failure	13
3.1.3.2 The Proposed Mechanism of the Initial Failure	13
3.2 Subsequent Retrogressive Failure	14
4.0 CONCLUSIONS	15
5.0 RECOMMENDATION FOR 1991 FIELD WORK	16
REFERENCES	18
TABLES I TO IV	19
FIGURES 1 TO 21	23
APPENDIX A	44
DISTRIBUTION	last page

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1.0 INTRODUCTION

On April 23 1990, a landslide occurred on the east bank of the Nipigon River 8 km downstream of the Alexander GS. The location, and the extent of the slide area are shown in Figure 1. This slide is referred to as the "1990 failure".

Subsequent to the slide, two soil investigations were conducted by Trow Ontario Limited between May 12 to May 23 1990. The initial investigation was done for TransCanada Piplines and the second investigation was done for the Ministry of Natural Resources. Between the two investigations, a total of seven boreholes were put down in the area (Figure 2) to determine the subsoil, and the groundwater conditions. Soil samples were also obtained for laboratory testing.

Based on their investigation, it was concluded that the following factors contributed to the 1990 failure/1/:

1) Toe erosion and slumping of the river bank.
2) Higher than normal moisture content in the soils.
3) Weak and sensitive soil layers in the slope.
4) Drawdown effect by river level fluctuations.
5) Tree harvesting to the northeast of the failure area.
6) Cleared right-of-way constructed by TransCanada Pipelines.

A study was initiated at the request of the Civil Maintenance Department to assess the impact of river level fluctuations caused by the Alexander GS operation on the downstream banks of Nipigon River. The impact of river level fluctuations on the bank slope stability depends upon the slope geometry, the soil conditions and the groundwater regime present in the bank slope. While the river level fluctuations may have been one

of the several contributing factors to the initial bank slope failure, the retrogression of that failure to a large scale landslide is mainly due to land controlled factors such as site geology, subsurface conditions, drainage pattern, and land cover. Since very limited data was available on these factors to assess the impact of river level fluctuations relative to other land controlled factors, a comprehensive study was undertaken in two phases:

Phase 1: A literature review and airphoto study of the area were conducted to determine the site geology, drainage pattern, land forms and land use, and as well as to identify the presence of any old bank failures or land slides in the area. This work was carried out by Bird and Hale Ltd. A report on their findings was produced independent of this report/2/.

Phase 2: In this report, the results of a review of the available geotechnical data and a parametric analysis of the 1990 failure were presented. An assessment was made of the relative importance of the factors that may have caused the 1990 failure. The results from the field measurements by the Hydraulic Study Group regarding the relationship between discharge rate and river level fluctuation, and river channel profile were made use of in this report. A field checking program was proposed to confirm several assumptions made in the parametric slope stability analysis and airphoto interpretation. The nature and extent of this field testing and monitoring program were outlined in the conclusion of this report.

After field checking a reanalysis of the slope stability and the retrogressive failure will be made to assess the effect of river level fluctuations on the bank stability and to identify if a potential exists for future slope failures of similar magnitude along Nipigon River.

2.0 THE SITE CONDITIONS

The Nipigon River runs from Alexander GS to Lake Helen, generally in a southerly direction with several bends along its length. In the post-glacial era, the valley was inundated by water from Lake Minong and Lake Houghton, the predecessors of Lake Superior. During this period, a thick layer of lacustrine sediment was deposited in the valley. With the crustal rebound and retreat of Lake Superior shoreline to its present location, a series of changes of glacial lakes and their drainage pattern occurred in the area.

Aerial photographs of the valley indicate numerous poorly drained areas with organic soils covering the lacustrine deposit. The soil conditions along the river are largely made up of varved silt and fine sand with

some clay layers. The soils at the ground surface are slightly coarse grained as compared to the soils at depth. In addition to the lacustrine deposit which fill the valley, there are high terraces of coarse grained soils and tills with rock outcrops to the east and west of the river valley.

The exact nature of the lacustrine soils present throughout the valley is not known in detail, however, the terrain conditions indicate some uniformity along the length of the river under examination as well as along the shores of Lake Helen. An airphoto study of the area indicated a number of erosional features and slope failures which are largely located in the lacustrine sediments, along the river banks south of the railway bridge.

2.1 Bank Slopes and Site Drainage

The terrain adjoining the Nipigon River ranges from almost flat in the areas of organic and lacustrine soils to near vertical slopes in some rock outcrop areas. Deep gullies are cut into the slopes where the tributary streams join the Nipigon River. Steep terrace-like features are also noticed at the boundary of old failure scarps.

The land adjoining the Nipigon River is drained by a network of short tributary streams to the main channel, in some cases, into the uplands on either side of the river. In the areas covered with organic soils, the frequency and depth of the drainage channel are low.

The soil types exposed along the river bank where erosion is taking place are consistently lacustrine in origin, and largely silts, clays and fine sands. Clay rich soils which are less erodible than the silts or fine sands are present as terraces along the base of river banks. In many areas, the lacustrine deposits are covered with fine sandy soils which exhibit a high capillarity. Relatively flat topography and the soil capillarity contribute to the accumulation of organic soils over large stretches of the lacustrine material. These areas also exhibit a high degree of infiltration relative to those steeply sloping areas.

2.2 Past Slope Failures

Based on airphoto interpretation, Bird and Hale Ltd. have identified scars of at least 25 retrogressive type failures of various sizes, (Table I). They report that about 77% of the river banks have experienced a single event slump. Some estimates of erosion rates at specific sites were made in their report, which showed that

most persistent and rapid erosion occurred on the outside downstream corners of the river. They also concluded that there has been a significant increase in the number of multiple event (retrogressive) failures in the past 10 to 15 years. These are concentrated in the lower reaches of the river from location 7.3 to 10.4 km on both sides of the river, with steeply sloping, and previously eroded banks.

Their study showed that the following characteristic features are common to most retrogressive failures in the area:

(a) Actively eroding river bank.
(b) Steeply sloping river bank.
(c) River access to toe of the bank is soil and not rock.
(d) Tributary streams at right angles to the river channel.
(e) Bank heights in excess of 3 m.
(f) Lacustrine soils consisting of layered silts, fine sands and clays.
(g) Mineral soils at the ground surface near the river bank.
(h) Organic soil overlying mineral soil in the watershed of failure area.

2.3 The 1990 Failure Site

The topography of the 1990 failure site prior to failure consisted of a plain dissected by several streams trending at right angles to the Nipigon River. In the interior of the site, the stream channels were relatively shallow in the order of 1 metre or less. Towards the river bank, the streams were more deeply incised, and at the point of entry to the river, the tributary bank heights were similar to the river bank heights, and the topography was quite convoluted (Figure 3).

An estimate of the pre-slide topography of the 1990 failure site was made by using airphoto coverage of 1979. The profile of the river bank at four locations in this area was estimated as shown in Figures 3 and 4. The accuracy of this estimate was limited by the masking effects from the tree canopies, bank shadows and leaf coverage on the topography features to be mapped. However, Figure 4 shows that there were steep bank slopes approaching 1(v):1.5(h) in the 1990 failure area, with the toes of the slopes regularly being inundated by the river. The height of the banks was estimated to range from 6 to 10 metres.

Most of the watershed for the 1990 failure site is relatively flat with the exception of a narrow strip along the northern boundary. Well defined stream channels are confined to the lower reaches. These characteristics,

combined with the organic soils which tend to absorb surface water, indicate a watershed with relatively high infiltration rates.

In 1931, the 1990 failure site and its watershed were covered with either forest, or regenerating forest. Prior to 1952, about 40% of the watershed, including some of the failure site, was clear cut. Subject to the limitations of non-stereo, small scale photographs, no retrogressive failures were identified in this area in 1952, and only one small persistent single event site was found in 1931, 1952, and 1962, by which time the area had regenerated significantly. Since that time, the only other significant change was the large clear cut area which appears on the 1990 photographs. About 40% of the watershed has been cleared.

The cleared TransCanada Pipelines right-of-way was visible for the first time on the 1962 photographs. It appears as though the soils through which the pipeline is constructed north of the watershed are relatively coarse and permeable, and thinly layered over bedrock. The pipeline trench would act as conduit if it was deep enough to penetrate bedrock. This is possible, but cannot be determined by airphoto interpretation without field work. Assuming that this was the case, it is estimated that the watershed area of the 1990 failure site would have been increased by about 50%.

The gravel extraction site which is first seen on the 1952 photograph, may have been present in 1931 but could not be confirmed because of the lack of airphoto coverage in that area.

2.3.1 Soil stratigraphy

The soil stratigraphy at the 1990 failure site is interpreted from the borehole logs of the soil investigation report by Trow Ontario Limited/1/. The soil deposit is divided into 4 units as: 1) Upper Silty Sand, 2) Clayey Silt, 3) Lower Silty Sand, and 4) the interbedded Silt and Clayey Silt. The Atterberg Limits of the Clayey Silt and the interbedded Silt and Clayey Silt are listed in Table II.

Based on its index properties, the strata (unit No. 2) previously identified in Trow's report as clayey silt is reclassified as a low plastic silty clay (CL, Figure 5). For the purposes of slope stability analysis, this silty clay layer is subdivided into two units; the upper firm silty clay and the lower soft silty clay. The interbedded silt and clayey silt (unit No. 4) is fairly deep seated and is generally below the river level. From the results of the field vane tests, and the SPT blow counts, the material properties of the different soil units are assumed as follows:

	Friction Angle degree	Cohesion (kPa)	Unit Weight (kN/m^3)
Upper silty sand*	30	0	17.6
Upper silty clay - firm*	28	30	19.0
Lower silty clay - soft*	28	0	19.0
Lower silty sand**	35	0	17.6
Interbedded silt and clayey silt**	23	0	19.0

* Assumed in Trow's report.
** Assumed by us

2.3.2 Geometry of the Slope

The bank geometry of the area prior to the landslide determined from the airphoto study is shown in Figures 3 and 4. The river bank in the failure area is located on the river bend and has been subjected to active erosion by the river. Also, the bank shape is irregular and dissected by streams joining the Nipigon River. The accuracy of this estimate was limited by the masking effects from tree canopies. Therefore, the slope profile above the river level determined by Trow's geotechnical investigation was used for this analysis. The geometry of the bank slope and the land topography given in Trow's report are presented in Figure 6 in which the bank slope has an average inclination of 1(v):1.15(h) but the portion of the slope submerged in the river has an inclination of 1(v):10(h). In this study, the river channel slope of 1(v):1.284(h) is used to extend the part of the bank slope below the river level based on a 1990 hydraulic test study at Station SB7, slightly north of the failure area. The ground surface profile of the area behind the crest of the slope shown in Figure 7 was extrapolated from the ground elevation of the boreholes. The benchmark used in Trow's report was arbitrary. All the elevation values used in Trow's report are adjusted to CGD by using the minimum river level established from the hydraulic test study as the reference/3/.

2.3.3 River Levels and Drawdown

The river level at this site is controlled by the discharge at the Alexander GS. In September of 1990, a hydraulic flow test was conducted by the Department of Civil Engineering & Architecture to evaluate the effect of river fluctuations on fish spawning. Figure 8 shows the rate of discharge at the Alexander GS and the river level response at different measuring stations downstream of the Alexander GS/3/. The river level data from Station SB7 which was the closest to the slide area is used for this study. Figure 9 shows a

relationship between the rate of discharge at the Alexander GS and the river level at Station SB7. From this plot, the maximum and minimum river levels at the failure area are estimated to be 185.7 m and 183.7 m corresponding to the maximum and minimum discharge rates of 540 and 113 cms.

Figure 10 shows a normalized plot of the river level versus time. It indicates that the river level at Station SB7 has a response time (or lag time) of approximately 1 day for the flow changes at Alexander GS (Approximately 50% of the river level change occurs in 3 to 4 hours, and 90% of the change occurs in 12 to 20 hours.) The hourly discharge rates from January 1985 to April 1990 show that the discharge rates fluctuate rapidly/3/. In order for the drawdown condition to manifest at the site, a period of 1/2 to 1 day of low rate of discharge preceded by a high rate of discharge is required.

The hourly discharge rate versus time plot for the month of April 1990 as presented in Figure 11 suggests that a drawdown condition was present prior to the slide. The magnitude of the drawdown is estimated to be 0.6 m (from 184.4 to 183.8 m). The rate of this drawdown is about 0.1 m/hr for the initial 50% of river level fluctuation and 0.002 m/hr for the remaining portion. It should be noted from the historic river discharge data that similar drawdown events have occurred many times in the past without the occurrence of large landslides.

2.3.4 Groundwater Conditions in the Bank Slope

The groundwater regime in the river bank is controlled by the soil stratigraphy, the recharge boundary behind the slope, and the discharge boundary condition at the bank slope. The piezometric levels presented in the Trow's report are shown in Figure 7. These measurements were done about one month after the failure. The actual piezometric levels prior to the failure could have been much higher. The groundwater regime and the pore water pressures in the slope prior to the 1990 failure are estimated by constructing a seepage flow net using the following assumptions:

(1) The phreatic surface corresponds to the piezometric levels recorded by Trow's geotechnical investigation one month after the landslide.

(2) The river level prior to the drawdown condition is at elevation 184.4 m. Corresponding to a discharge rate of 250 cms at Alexander GS.

(3) The effects of stratification of the deposit are not considered in the construction of the seepage flow net for the reasons of simplicity and not having the detailed soil properties.

3.0 THE POSTULATED FAILURE MECHANISM

Based on the kinematic considerations of the aerial dimensions of the failure geometry (Figure 2), it is postulated that the 1990 failure occurred in two stages: 1) the initial slide of the bank slope, and 2) the subsequent retrogressive failure of the land behind the bank slope.

Factors that may have contributed to the 1990 failure are:

- slope geometry.
- soil conditions.
- groundwater conditions.
- river level.

1) Slope geometry - The failure was located on the outside bank of the river where scouring of the river bank and toe erosion was going on. Also the fluctuation of the river level causes cyclic wetting and drying of the soil and induces slumping of the bank. These processes can transform the slope profile to a more critical configuration and thus lead to an initial slide of the slope. A bank erosion rate of 0.5 to 1 m/year is estimated from airphoto studies/2/.

2) Soil conditions - different types of failure mechanisms dominate in different soil types. For example in Eastern Canada, retrogressive type of slope failures are often reported to occur in sensitive clay deposits/4/. At this site the presence of the soft and low plastic silty clay layer combined with the possible high groundwater pressure within the lower silty sand layer might have led to a retrogressive type failure.

3) Groundwater condition - the stability of the slope is a direct function of the pore water pressure within the slope. Any increase in pore water pressure due to the changes in hydraulic boundary conditions decreases the stability of the slope.

Above the phreatic line, the soils exhibit suction. This suction caused by the capillary action induces negative pore water pressure in the soil and thereby increases the effective shear strength of the soil. The change of soil moisture from partially saturated to saturated condition destroys the capillary or matric suction in the soil and reduces the stability of the slope/5/.

4) River level - The change in river level can have dual effects on the stability of the slope:

(i) The rise in river level acts as a loading on the toe to improve the stability of the slope. With the lowering of the river level, the hydraulic load on the toe is reduced and the stability of the slope decreases.

ii) The rapid lowering of the river level can create a sudden drawdown condition (a condition in which the phreatic line has no time to adjust to the new river level and reduce the stability of the slope. In terms of the bank slope stability the rapid rise in river level has the same effect as the slow rise in river level.

In the next section, the effects of each of these factors on the initial slide are examined by a parametric slope stability analysis. Then the conditions that existed in the site prior to the 1990 failure are discussed.

3.1 Initial Slide

3.1.1 Parametric Study

The slope geometry, soil conditions, groundwater conditions, and river level prior to the 1990 failure inferred by the available sources of information are presented in Table III. With the uncertainty involved in these parameters, a parametric slope stability analysis was carried out to determine the impact of these parameters on the initial bank failure and to rank them. From the results of this study, the type of information required to be determined in the 1991 field program is identified.

The commercial computer program PC-SLOPE by GEO-SLOPE Programming Ltd. was used for this study/6/. (The option of Bishop's simplified method for slope stability analysis was used.) The effect of soil suction or negative pore water pressure was taken into account by invoking the soil suction option of the program. In this program the soil above the phreatic line is assumed to have full suction. In coarse grained soils, this assumption may overestimate the suction effect.

Based on the site conditions, the possible ranges of variation for each parameter were selected (Table IV). In each analysis, only one of the parameters in the Base case was changed. Cases 2 to 6 were analyzed to study the effects of varying the shear strength values of the upper and lower silty clay layers on the stability of the slope. The effects of soil suction above the phreatic surface on the stability of the slope were examined in Case 7. The effects of river fluctuation were considered in Cases 8 and 9. In Case 8, the maximum river level, 185.7 m, was used. In Case 9, a river level of 183.8 m corresponding to a discharge rate of 125 cms at Alexander GS was used. For these two cases, the groundwater flow pattern of the Base

case (Figure 12) was modified to take into account the changes in the hydraulic boundary condition at the toe of the bank imposed by the changes in the river's level. The modified groundwater flow patterns, shown in Figures 13 and 14, are for steady state equilibrium conditions with the new river levels. In Case 10, the effect of rapid river drawdown from elevation 184.4 m to 183.8 m on the stability of the bank was studied. This drawdown condition is similar to the drawdown event that occurred prior to the 1990 failure. The rapid drawdown of the river created a seepage surface at the face of the bank between elevations 184.4 and 183.8 m. The resulting groundwater flow pattern immediately after drawdown, which is in a transient state, is presented in Figure 15. With time, the phreatic surface adjacent to the toe of the bank would drop to its equilibrium position intercepting the bank slope face at elevation 183.3 m. At that time, the groundwater pressure pattern is identical to the groundwater pressure pattern of Case 9.

In Cases 11 and 12, the effects of changing the slope geometry by erosion on the bank stability is examined. In Case 11, the bank slope above the river level is flattened from 1(v):1.15(h) to 1(v):1.5(h) slope as shown in Figure 16. Such flattening of the bank may be caused by the surface erosion of the bank. The slope geometry used in Case 11 also corresponds with the results of the airphoto interpretation study. The scouring effect of the river could steepen the slope of the bank beneath the river level. In Case 12, the effect of steepening of the bank slope beneath the river from 1(v):2.84(h) to 1(v):2(h) (Figure 17) on the stability of the bank was examined.

The phreatic surface used in the Base case was based on the field data obtained one month after the occurrence of the 1990 failure. After the failure, some of the excess pore pressures might have been dissipated. Therefore, in Case 13, a higher phreatic surface was used to assess its effect on the bank slope stability. The groundwater flow pattern used in this study is shown in Figure 18. The phreatic surface was assumed to be located beneath the weathered zone of the soil. The contact between the weathered and the unweathered soil was determined by the changes in colour of the soil from brown to grey described in the borehole logs of Trow's geotechnical report.

3.1.2 Results of the Parametric Study

The results are presented in Table IV. The effect of each parameter on the stability of the bank slope is expressed as a percentage change in the factor of safety with respect to the Base case. The following is a brief discussion of the results.

Material Properties

Upper silty clay - The results from Cases 2 and 3 show that the shear strength properties of the upper silty clay have a minor effect on the bank slope stability. By decreasing the friction angle from 28 to 26 degrees, the slope stability decreases by 0.2%. A reduction in the c' value from 10 kPa to 5 kPa lowers the slope stability by 1.6%. As shown in Figure 12, the slip surface in the upper silty clay layer is steeply inclined and the contribution by this layer to the resisting force against failure is minimum. Therefore, the changes in shear strength of this layer have a minor effect on the overall stability of the slope.

Lower silty clay - Case 5 shows that the decrease in the c' value from 5 kPa to 2.5 kPa of the lower silty clay decreases the slope stability by 8.7%. This change in the factor of safety is associated with the changes in the failure mode from a deep seated failure to a shallow seated failure (Figure 12). In the shallow seated failure mode, most of the resisting force against failure comes from the lower silty clay layer. Therefore, any changes in the c' value will result in a significant change in the stability of the bank slope. The results from Cases 5 and 6 also show that the percentage reduction in the safety factor of the bank slope is directly proportionate to the reduction in the c' value. In Case 6, reducing the c' from 5 kPa to 0 kPa results in a reduction of the factor of safety by 16.7% double that for Case 5.

The reduction in the friction angle value of the lower silty clay layer also alters the failure mode of the slope from a deep seated failure to a shallow seated failure. Reducing the friction angle value by 2 degrees reduces the factor of safety by 3% (Case 4).

Soil suction - The effect of soil suction on the calculated factor of safety is demonstrated by the result of Case 7. By eliminating the soil suction effect, the significant decrease in slope stability of 9.9% is estimated.

River level - The increase in the river level from 184.4 to 185.7 m improves the stability of the slope by 1.5% (Case 8). This effect is not as significant as the decrease in the river level from 184.4 to 183.8 m as shown in Case 9 resulting in a decrease in stability of 7.9%. This is because, with the increase in the river level,

the phreatic surface adjacent to the toe of the bank rises, and destroys the soil suction. This effect has been taken into account in Case 8.

Rapid drawdown - The effect of rapid drawdown of the river from 184.4 to 183.8 m is to decrease the factor of safety of the slope by 9.2%. This decrease in the factor of safety of the bank is caused by the combined effect of lowering of river level, and the excess pore water pressure created in the slope by the rapid drawdown. The excess pore water pressure effect can be calculated by subtracting the factor of safety of Case 9 from that of Case 10. A decrease in the factor of safety by 1.3% was calculated.

Slope geometry - The effects of slope geometry on the bank stability are illustrated by the results of Cases 11 and 12. In Case 11, the flattening of the bank slope from 1(v):1.15(h) to 1(v):1.5(h) improves the stability of the slope by 9.7%. The steepening of the submerged portion of the bank from 1(v):2.84(h) to 1(v):2(h) decreases the stability of the bank slope by 5.4%. These results indicate that the slope geometry has a significant effect on the stability of the bank.

Groundwater pressure - The change in the maximum phreatic surface elevations from 186.9 m to 190.7 m, Case 13, reduces the factor of safety of the bank by 14.5%. The effects of raising the phreatic surface are twofold: 1) a general increase in pore water pressure, 2) a reduction in soil suction in the zone above the phreatic surface. By increasing the maximum phreatic surface from elevation 186.9 m to 190.72 m, the soil suction within the slope is almost completely eliminated. By comparing Case 7 and Case 13 (Table IV), the net effect of increasing the phreatic surface on the slope stability without considering the soil suction effect is only 4.6%.

3.1.3 The Proposed Failure Mechanism of the Initial Failure

The results of the parametric slope stability analysis indicate that the groundwater pressure, the shear strength of the lower silty clay layer, the loss of soil suction, the drawdown condition, the river level fluctuation, and the slope geometry have a very significant effect on the occurrence of the initial slide. In this section, the conditions that existed or occurred at the site prior to the failure are examined and the failure mechanism is postulated.

3.1.3.1 Conditions prior to the initial failure

The meteorological data for Thunder Bay area indicated that, 5 days prior to the failure, a warm spell occurred which caused the snow to melt and the ground to thaw. This warm spell could have had a threefold effect on the occurrence of the 1990 landslide, such as; 1) The runoff caused by the melting snow could have destroyed the soil suction in the upper silty sand layer. 2) The runoff water in the water shed area could have been intercepted and accumulated in the pipeline trench to act as a water filled tension crack and decreased the stability of the slope. 3) If the lower silty sand layer was connected to a recharge boundary at the clear cut area north east of the landslide area, a very high pore water pressure could have developed within this layer and propagated into the lower soft silty clay layer. With about a 40% watershed area or the recharge boundary for the landslide area cleared of trees, a rapid melting of snow due to solar radiation could have worsened the groundwater conditions.

The discharge rate record at Alexander GS shows that on April 19, 1990, the river level dropped from elevation 184.4 m to 183.7 m over a period of 24 hours, creating a drawdown condition at the toe of the bank slope.

3.1.3.2 The Proposed Mechanism of the Initial Failure

It is proposed that prior to the failure the river bank profile was in a relatively critical configuration caused by toe erosion and scouring of the river. Even though the river drawdown event had an effect on the slope stability of the bank, it alone could not have caused the 1990 initial failure, because the slopes in this area had experienced drawdown events of similar magnitude several times as indicated in Figures 19, 20 and 21 without any major slope instability. In addition, the failure occurred 4 days after the drawdown event. Therefore, additional factors must have existed for the failure to occur.

The warm weather began 5 days prior to the failure providing a ground thawing condition at the site. The snow melt water could have destroyed the soil suction in the upper silty sand layer, and increased the pore water pressure in the lower silty sand and the lower silty clay layer through a recharge boundary in the high-terrace area as indicated by the airphoto study. It is believed that the combined effect of a soil suction destruction in the upper silty sand layer and an increased pore water pressure in the lower silty sand and the lower silty clay layers provided the additional necessary condition for the occurrence of the failure.

3.2 Subsequent Retrogressive Failure

It is postulated that the 1990 failure was initiated by a slope failure at the river bank which retrogressed inland leading to a large scale landslide. A number of retrogressive type of failures have been recorded in very sensitive clays of Eastern Canada/7/ and Norway/8/. The mechanism of these failures have been analyzed by Tavenas et al/9/. Based on an analysis of several landslides in sensitive clays in Eastern Canada, they proposed 4 criteria to assess the risk and extent of retrogressive slide.

1) There must be an initial slope failure,
2) There must be a continued backscarp instability (Stability factor = $\gamma H/c_{uv} > 4$),
3) The slide debris must have the ability to flow when remolded (remolded shear strength $c_{ur} < 1$ kPa or liquidity index $I_L > 1.2$), and
4) The slide debris must have the ability to become remolded during the slide.

The results of a slope stability parametric analysis of the river bank at the 1990 slide area indicated that conditions were present for the occurrence of the initial slide, i.e. criterion 1 is satisfied. From the profile of the slide area, the height of the retrogressive zone is estimated to be 10 m for which $\gamma H/c_{uv}$ $19.0 \times 10/30 = 6.3 > 4$ and thus criterion 2 is satisfied. The remolded vane shear strength of the clay as given in Trow's report is 9 kPa which is much larger than 1 kPa specified in criterion 3. Also the liquidity index of the clay is 0.9 which is smaller than 1.2 specified in criterion 3. This means the failure debris could not flow when remolded.

It is possible that the mechanism of the retrogressive failure at this site might be different from that applicable to sensitive clays of Eastern Canada. The main difference being that the silty clay deposit encountered at the Nipigon River failure area is not as sensitive to disturbance as the quick clay of Eastern Canada to produce a retrogressive type of failure.

For this site it is postulated that with the initial slope failure, the drainage path for the lower silty sand layer at the bank face was blocked by the failed debris, causing a build up of a high hydraulic head. The maximum hydraulic head that could develop in this silty sand layer is equal to the elevation of the recharge boundary. This high hydraulic pressure could propagate into the lower soft silty clay layer and reduce its effective stress. The reduction in effective stress could in turn reduce the operative shear strength of the lower soft silty clay to a value smaller than 1 kPa required for the flow criteria to be satisfied. In addition, the height of the overburden above the lower silty sand decreases in the area where the retrogressive failure is occurring. Thus the total stress acting on the lower silty sand layer is reduced. Such a reduction in total stress, and the high pore water pressure may have been sufficient to create a liquefaction condition in the lower silty sand, and thereby cause the failure debris to flow to the river and facilitate further retrogression

to occur. This hypothesis can also explain why the majority of retrogressive failures identified by airphoto study were located in soft lacustrine deposits having high groundwater conditions with poor surface drainage patterns.

4.0 CONCLUSIONS

An airphoto study showed that the Nipigon River had a history of bank instability and retrogressive failures predating the construction of Alexander GS. A total of 25 sites of past retrogressive failure were identified of which 12 occurred between 1931 and 1990.

Most of the retrogressive failures took place in the outside banks at the river bends. These sites were concentrated in the lower reaches of the river 7.3 to 10.4 km downstream of Alexander GS. The 1990 failure site had most of the characteristics that were present at other retrogressive failure sites.

Based on the results of the parametric analysis of the initial bank failure, the following factors are ranked to be the most critical ones in the following order:

1. A high groundwater pressure regime existed in the site.
2. A loss of soil suction occurred above the phreatic surface due to surface infiltration from a rapid ground thaw.
3. There was a possible variation in the shear strength of the lower soft silty clay layer.
4. The slope geometry was altered by surface erosion.
5. There was a drawdown condition and a river fluctuation.

The high groundwater conditions and the loss of suction were both related to the rapid thawing of the ground and an infiltration of snow melt water that were present prior to slope failure.

The mechanism of retrogressive failure resulting in a large landslide is not well understood. The silty clay beds through which the failure occurred are not as sensitive as the clays of Eastern Canada and don't have the flowability required for a retrogressive type of failure.

It is hypothesized that the high pore water pressure in the silty sand layer underlying the soft sensitive clay bed caused liquefaction and allowed the failed debris to flow into the river. This hypothesis should be verified by a field investigation and an accurate soil property determination.

5.0 RECOMMENDATION FOR 1991 FIELD WORK

The results of the slope stability analysis and the postulated failure mechanism presented in this report are based on assumptions of the slope geometry, the shear strength of the soil layers and the pore water pressure conditions in the area prior to failure. Many of these important parameters had to be assumed or estimated on the basis of an airphoto study and limited soil investigations conducted in the area. Retrogressive type of failures similar to the 1990 failure have occurred along Nipigon River in the past as identified by a detailed airphoto study. In order to properly assess the role of the river level fluctuations and drawdown effects on the occurrence of such large scale landslides, more accurate information on the soil and groundwater conditions of the site are necessary. For this purpose, a field testing program is recommended to be carried out. An important component of this study is the determination of the pore pressure response of the soil at the toe of the bank slope to the river level fluctuations. Also, an assessment should be made of the impact of the river level fluctuation on the erosion of bank slopes (toe erosion) along the Nipigon River.

A soil investigation should be conducted to identify the different soil strata and their extent, for example the continuity of the lower silty sand layer and its recharge boundary. The index properties and shear strength properties of each stratum should be determined.

To identify the groundwater regime within the slope, a line of piezometers should be installed on a section of the slope located in an area close to but not influenced by the 1990 slide. To determine the effect of drawdown on the pore water pressure within the slope, it is recommended that fast response piezometers should be installed in the toe of a slope. The measured changes in pore water pressure corresponding to the changes in river level will provide an insight into the effect of the drawdown on the stability of the slope along the river.

Finally, it is recommended that several piezocone probings should be made in the failure area. The results of these probings will provide information on the location of the slip surface or the slip zone. A detail proposal of the field work program is outlined in Appendix A.

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POSSIBLE AND PROBABLE RETROGRESSIVE FAILURES ON THE NIPIGON RIVER

Feature	Location (km)	Date of Photo	Age of Failure	Visible on Photo 1931	1947	1952	1962	1975	1979	1990	*Width at Shore (m)	*Inshore Extent(m)	**Bank Height	Volume (m3)	Physiography at head	Associated with Streams	Associated with Recent Clearing
1	8.3 E	1931	old@	✓	✓	✓	✓	•	✓	✓	150	120	(8)	(140,000)	OL	yes	N/A
2	8.7 W	1931	old	✓							60	90	(8)	(60,000)	LP	yes	N/A
3	9.0 W	1931	old	✓	✓		✓	✓	✓	✓	160	400	(8)	(500,000)	OL/LP	yes	N/A
4	9.5 E	1931	old	✓	✓					✓	160	350	(6)	(500,000)	OL	no	N/A
5	9.7 E	1931	old	✓					•	✓	80	100	(6)	(50,000)	OL	no	N/A
6	9.9 E	1931	recent	✓	✓	✓	✓	✓	•	✓	80	50	8	90,000	OL	no	yes
7	10.9AE	1931	old	✓						✓	120	60	(2)	(30,000)	OL	no	N/A
11(47)	8.5 E	1947	old		✓		✓				200	200	4	200,000	OL	yes	N/A
8	1.4 W	1952	old			✓	✓	✓	✓	✓	120	100	16	250,000	LPI	yes	N/A
9	7.4 E	1952	old			✓				✓	50	20	(5)	(5,000)	LP	no	N/A
10	8.1 E	1962	old				✓	✓	✓	N/A	125	70	(8)	(80,000)	OL	yes	N/A
SITE 11 - was originally seen on 1962 coverage; recently acquired 1947 coverage show the same feature																	
12	8.3 E	1975	recent					✓	✓	N/A	110	60	6	50,000	OL	yes	no
13	1.4 E	1979	recent						✓	✓	130	30	8	30,000	LP	yes	yes
14	8.9 W	1979	recent						✓	✓	70	60	7	30,000	LP	yes	no
15	9.6 E	1979	recent						✓	✓	80	50	7	30,000	OL	no	no
16	9.9 E	1979	recent						✓	✓	80	50	8	30,000	OL	yes	no
17	10.0 E	1979	recent						✓	✓	100	30	5	15,000	OL	yes	no
18	0.8 E	1990	old							✓	450	200	6	500,000	LP/LPI	no	N/A
19	3.3 E	1990	old							✓	350	50	6	60,000	LPI/RO	yes	N/A
20	7.3 E	1990	old							✓	130	300	5	250,000	OL/RO	no	N/A
21	7.9 E	1990	recent							✓	385	430	10	1,200,000	OL/HT	yes	yes
22	0.4 W	1990	recent							✓	70	50	4	15,000	LP	yes	no
23	9.6 E	1990	recent							✓	70	50	5	20,000	LP	no	no
24	10.3 W	1990	recent							✓	200	60	4	40,000	OL/LP	no	no
25	7.1 E	1990	recent							✓	60	60	10	30,000	LP	yes	no

* measurements made on originally interpreted airphotos
** by parallax bar except where bracketed
() - estimated values
• site of reoccurring failure
@ old means prior to the Date of Photo

Table I Retrogressive failures on Nipigon River identified by airphoto study/2/

SAMPLE	L.L. (%)	P.L. (%)	P.I. (%)	L.I.	M.C. (%)	% SAND	% SILT	% CLAY
BH-1 5' - 7'						25	71	4
BH-1 10' - 12'	45.5	24.7	20.8	0.33	31.5			
BH-1 16' - 18'	39.0	18.9	20.1	0.97	38.4	2	74	24
BH-1 40' - 42'	22.5	15.7	6.8	2.87	35.2	1	69	30
Silt Block						30	64	6
Clay Block						4	68	28

Table II Summary of index properties of soils/1/.

MATERIAL PROPERTIES

	Density, kN/m3	ϕ', degree	c', kPa
Upper silty sand	17.6	30	0
Silty clay			
Upper	19.0	28	10
Lower	19.0	28	5
Lower silty sand	17.6	35	0
Interbedded silt & clayey silt	19.0	23	0

RIVER LEVEL : 184.4 m

GROUNDWATER CONDITION : Refer to Figure 12

AVERAGE BANK SLOPE

Above river level 183.8 m : 1 vertical to 1.15 horizontal

Below river level 183.8 m : 1 vertical to 2.85 horizontal

MAX. PHREATIC SURFACE ELEVATION : 186.9 m

Table III Data used for Case 1 - Base case.

CASE	F.S.	% increase in F.S. w.r.t. Case 1	PARAMETER CHANGES FROM CASE 1
1 (Base case)	0.919	----	----
2	0.917	-0.2	decrease ϕ' of the upper silty clay layer from 28 degree to 26 degree
3	0.904	-1.6	decrease c' of the upper silty clay layer from 10 kPa to 5 kPa
4	0.891	-3.0	decrease ϕ' of the lower silty clay layer from 28 degree to 26 degree
5	0.839	-8.7	decrease c' of the lower silty clay layer from 5 kPa to 2.5 kPa
6	0.784	-16.7	decrease c' of the lower silty clay layer from 5 kPa to 0 kPa
7	0.828	-9.9	soil suction effect eliminated
8	0.933	+1.5	river level increase from 184.4 m to 185.7 m (discharge from 250 to 540 cms)
9	0.846	-7.9	river level decrease from 184.4 m to 183.8 m (discharge from 250 to 125 cms)
10	0.834	-9.2	rapid drawdown - river level drop from 184.4 m to 183.8 m
11	1.008	+9.7	avg. bank slope (above river level 183.8 m) flatten from 1(v) : 1.15(h) to 1(v) : 1.5(h)
12	0.869	-5.4	avg. bank slope (below river level 183.8 m) steepen from 1(v) : 2.84(h) to 1(v) : 2.0(h)
13	0.786	-14.5	the max. phreatic surface level increase from 186.9 m to 190.7 m

Table IV Results of parametric study.

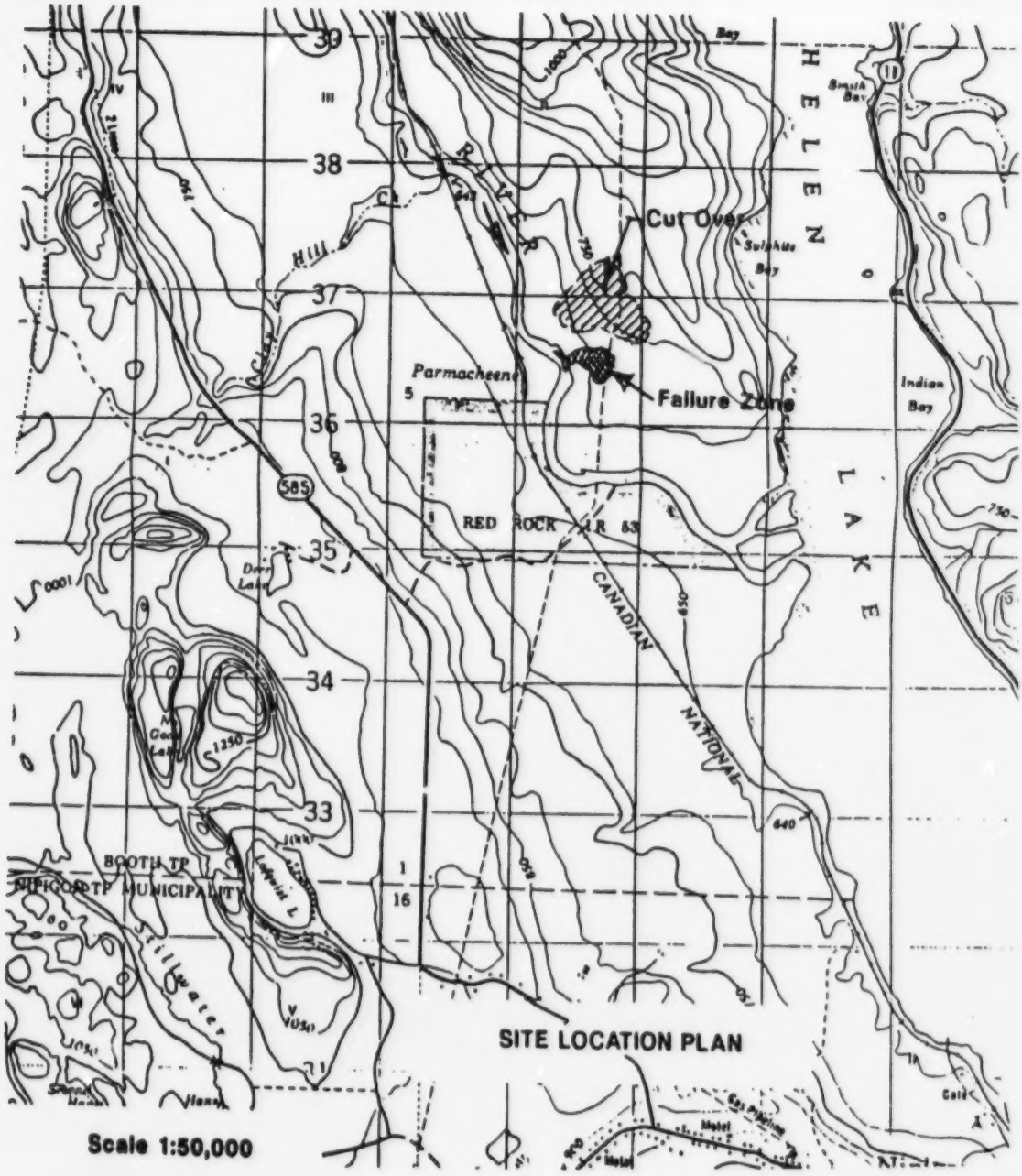
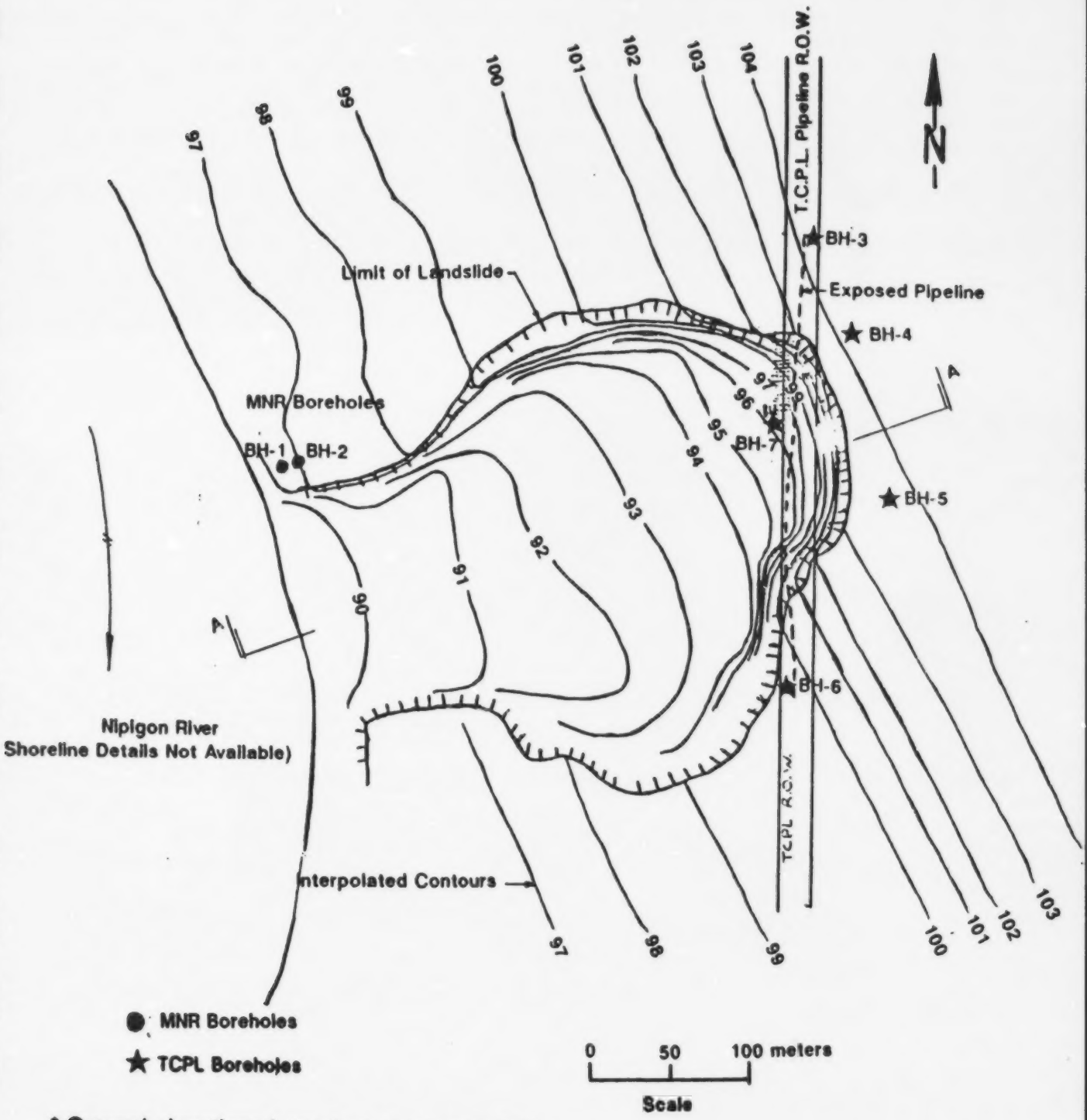


Figure 1 Site location plan /1/.



* Ground elevations based on temporary B.M. not w.r.t CGD

Figure 2 Site plan /1/.

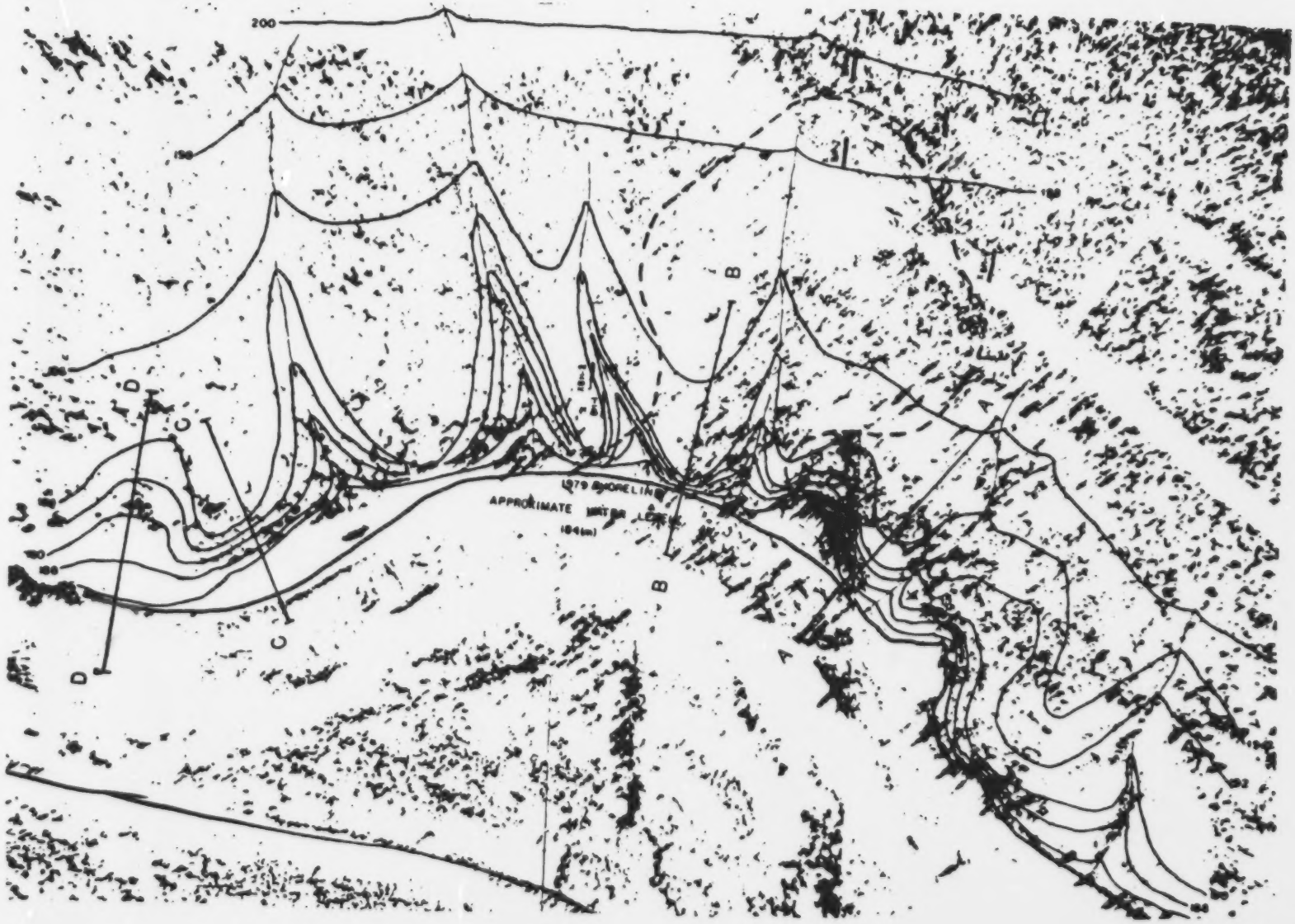


Figure 3 Pre-slump form lines - 1990 failure site/2/.

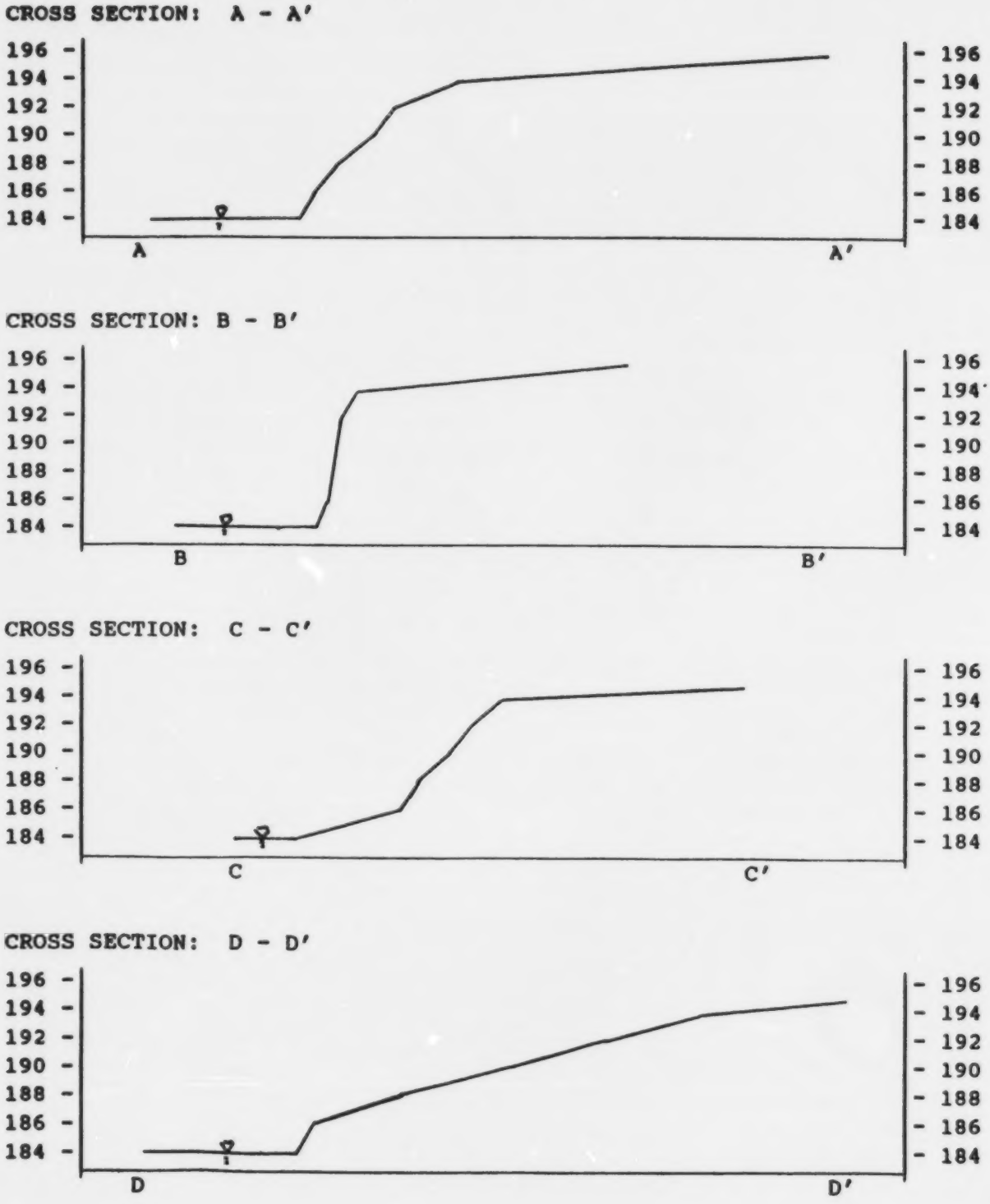


Figure 4 Cross sections at 1990 failure site/2/.

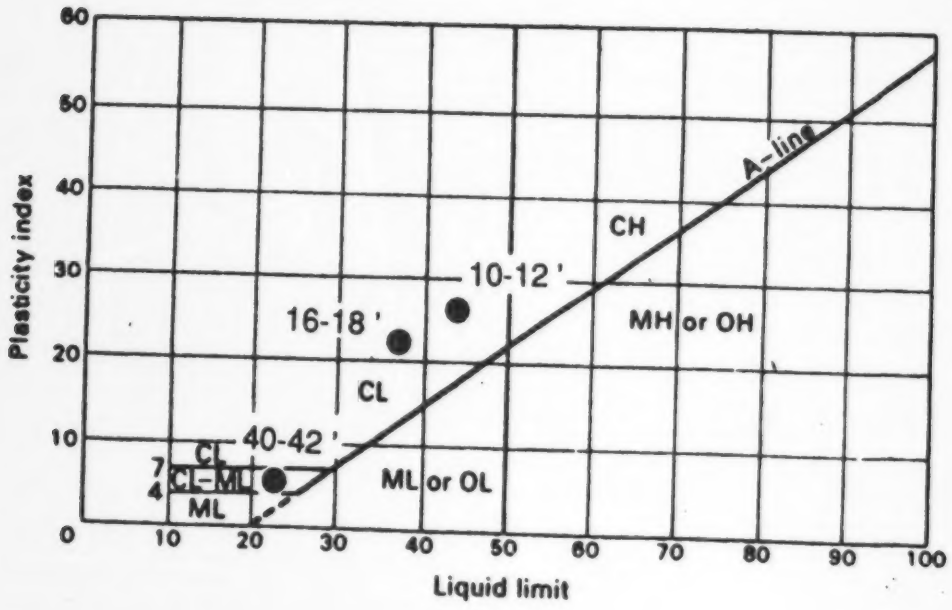


Figure 5 Soil classification chart by Atterberg Limits.

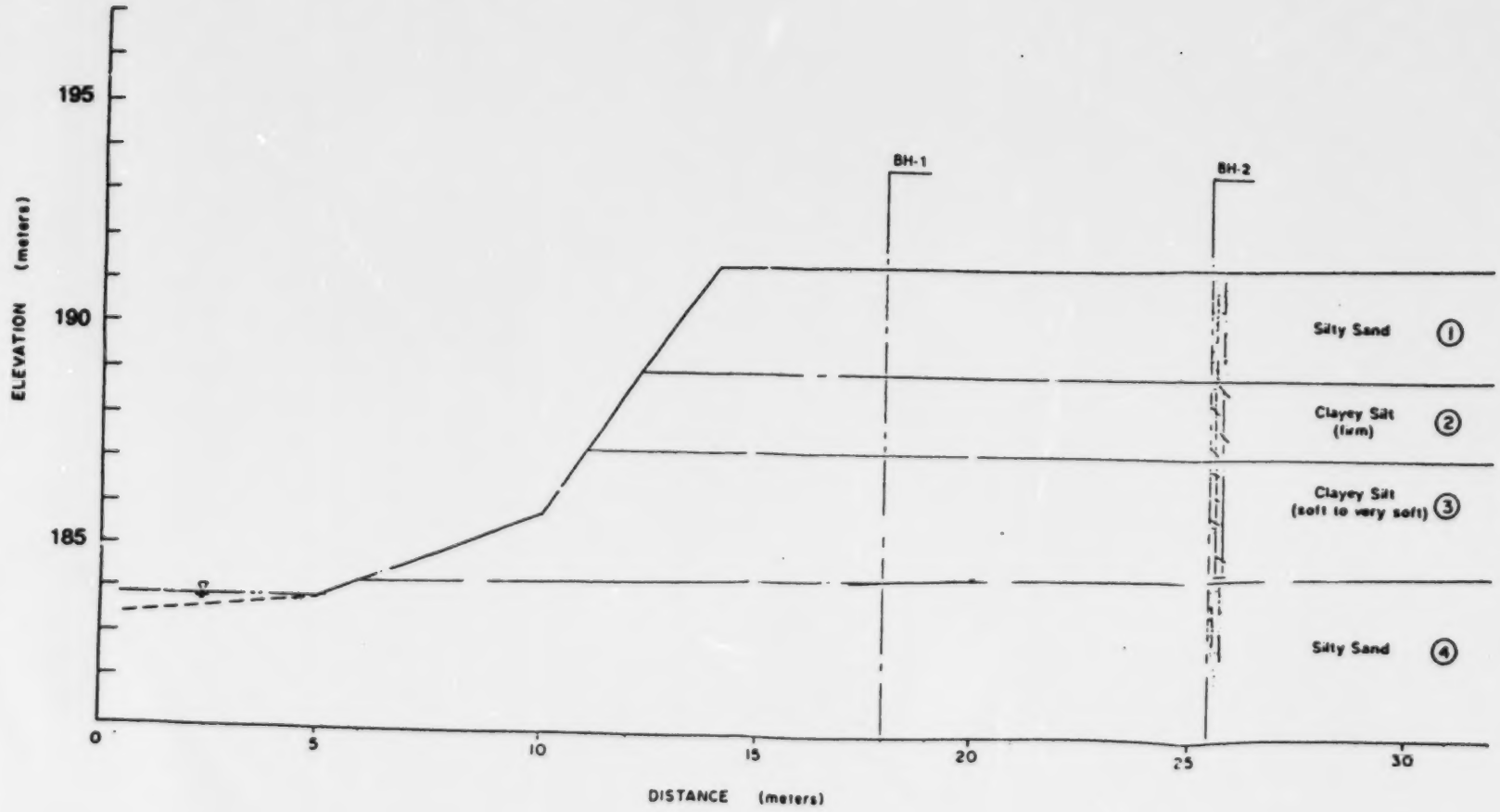


Figure 6 Bank slope profile/1/.

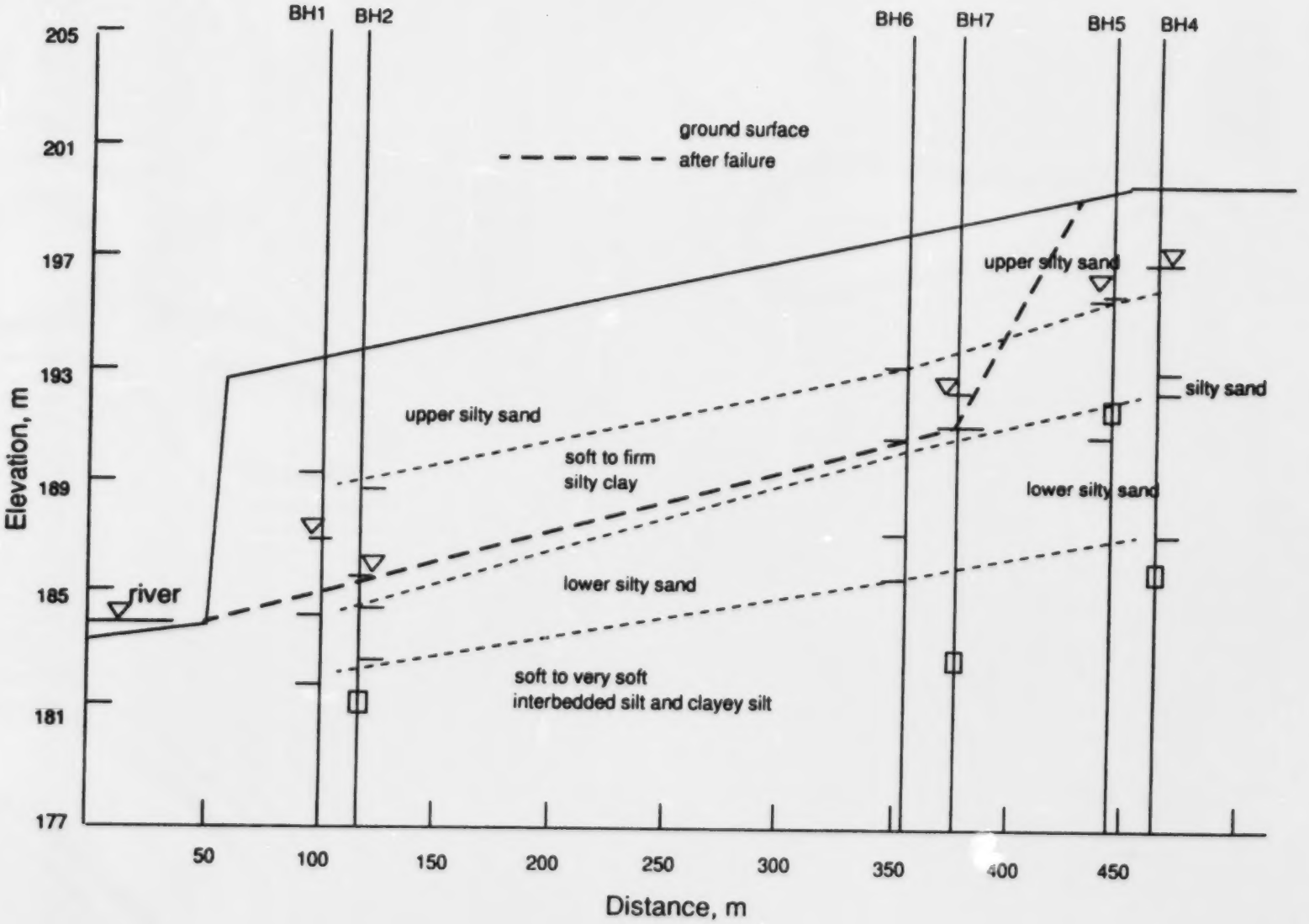


Figure 7 Slope profile based on the Trow's Borehole data.

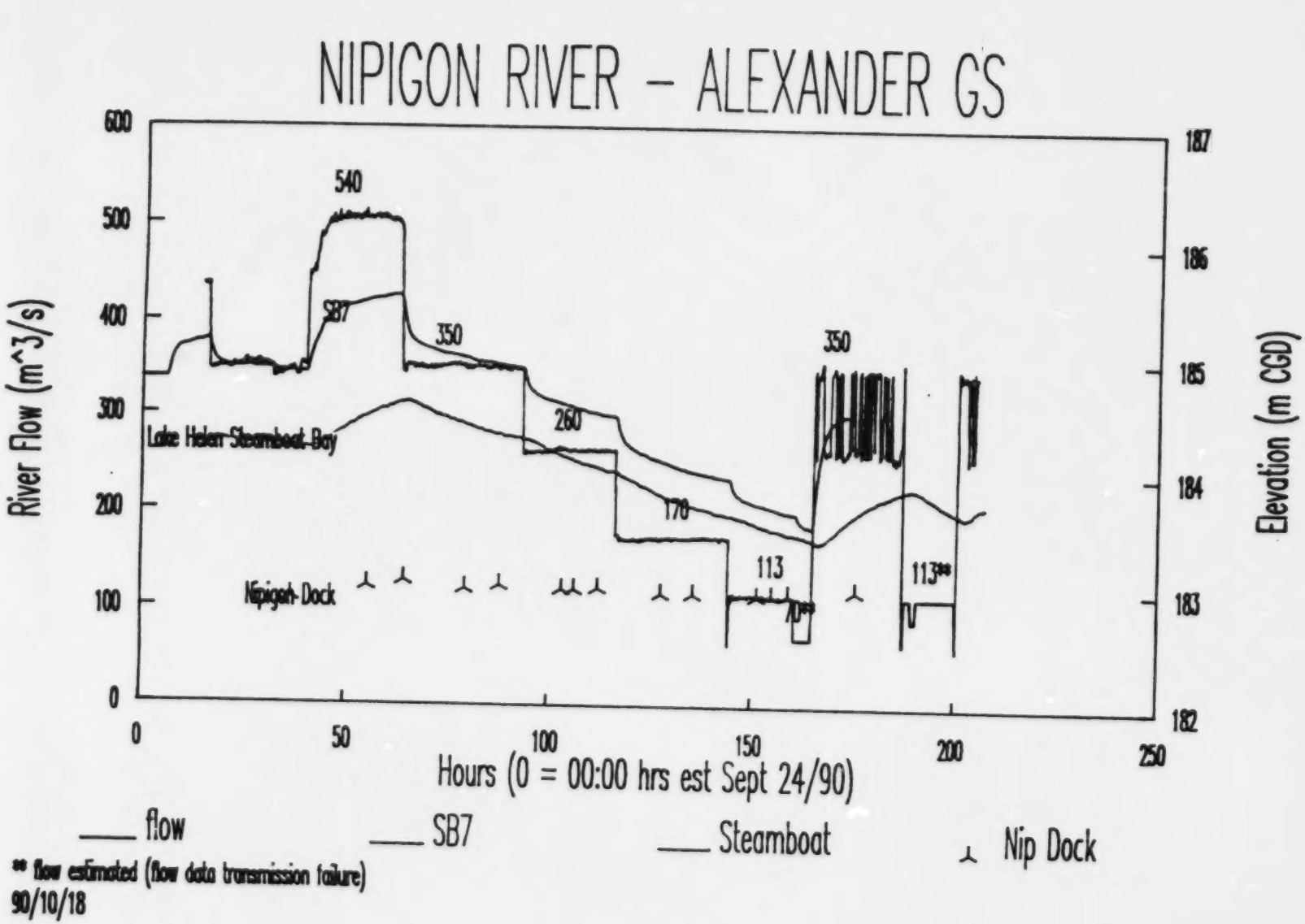


Figure 8 Results of the hydraulic test conducted by the Department of Civil Engineering & Architecture.

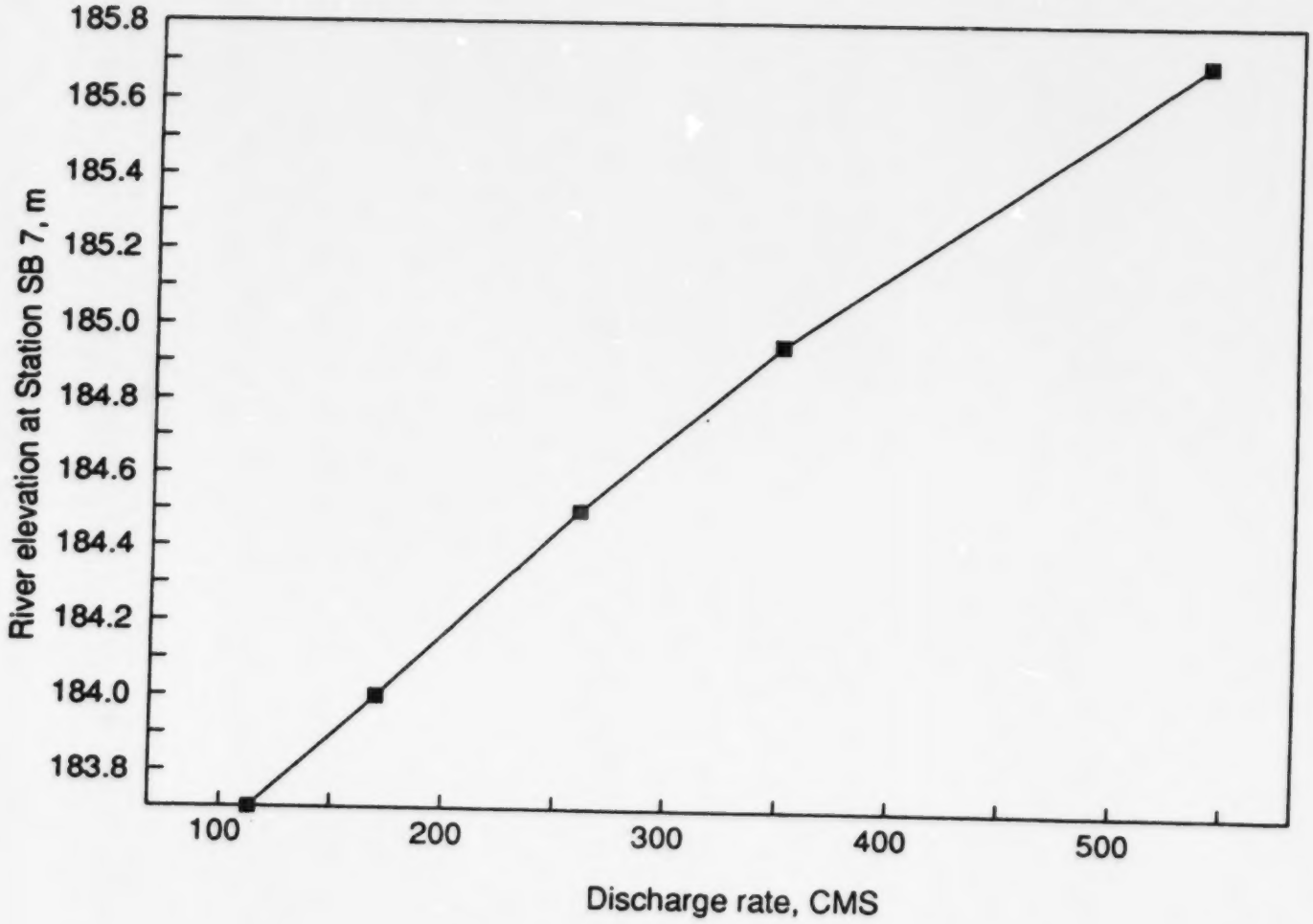


Figure 9 Rate of discharge at Alexander GS vs. river elevation at SB7.

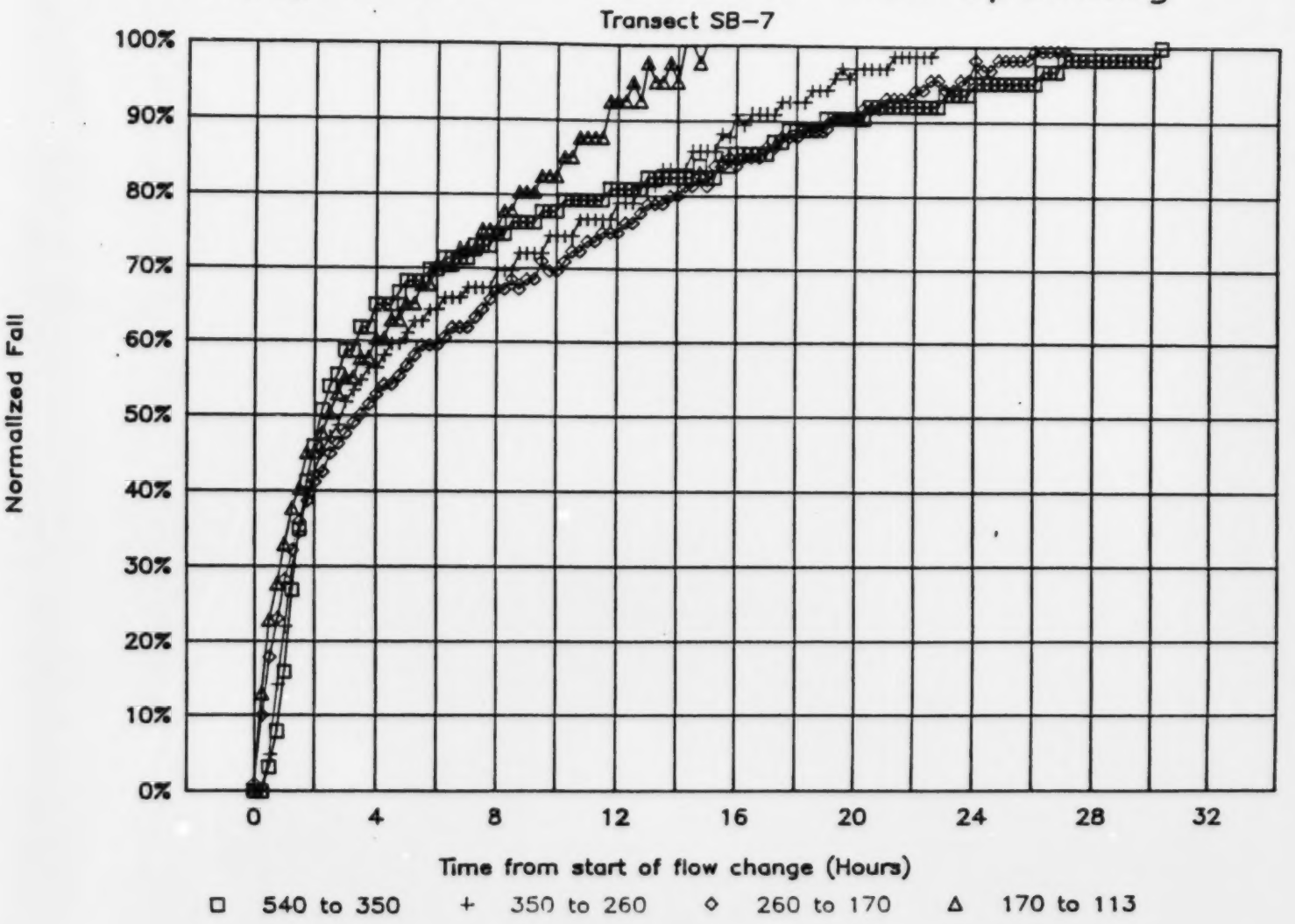


Figure 10 Normalized results of the hydraulic test conducted by Department of Civil Engineering & Architecture.

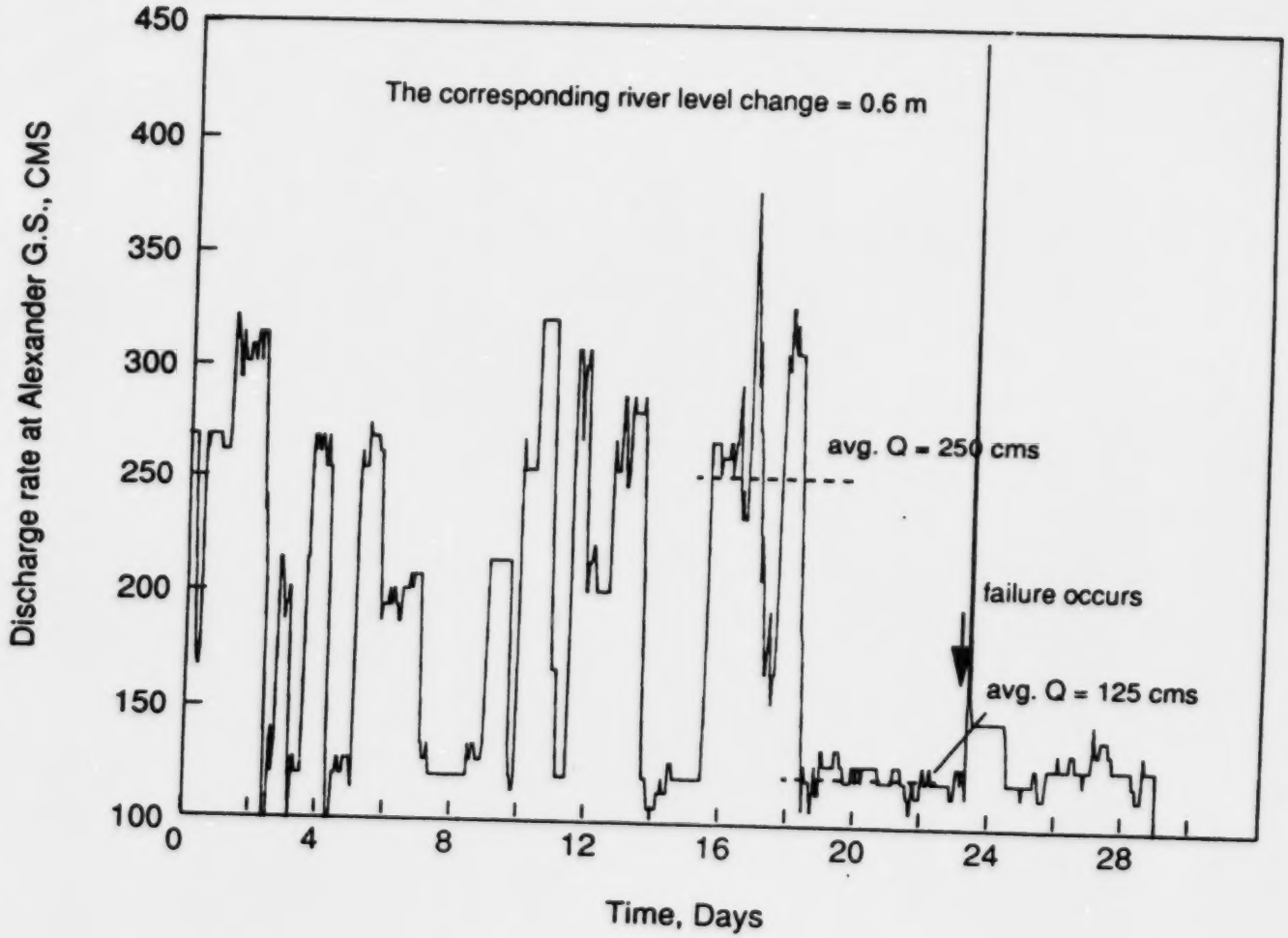


Figure 11 The hourly discharge at Alexander GS - April 1990.

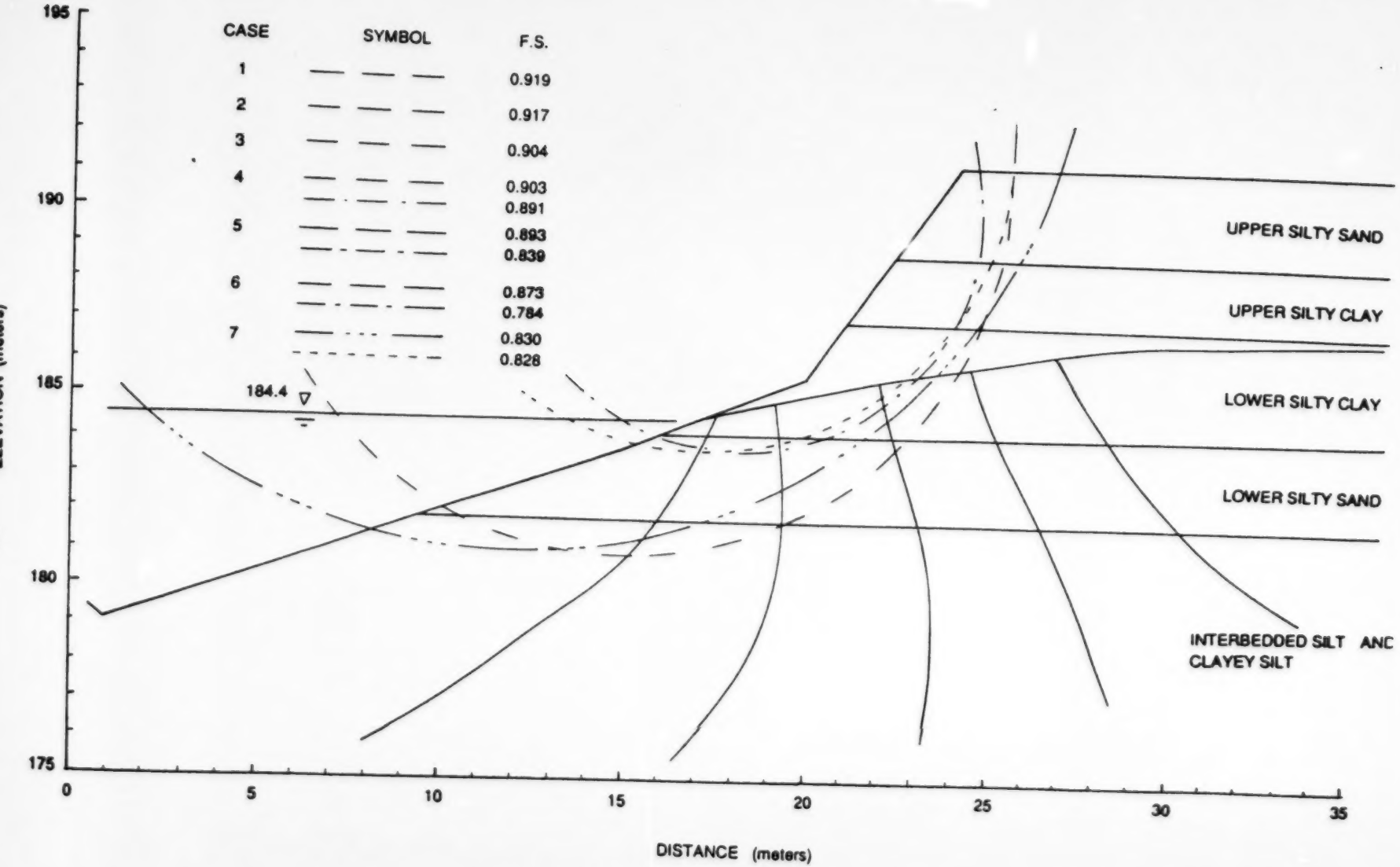


Figure 12 Slope stability analysis - Cases 1 to 7.

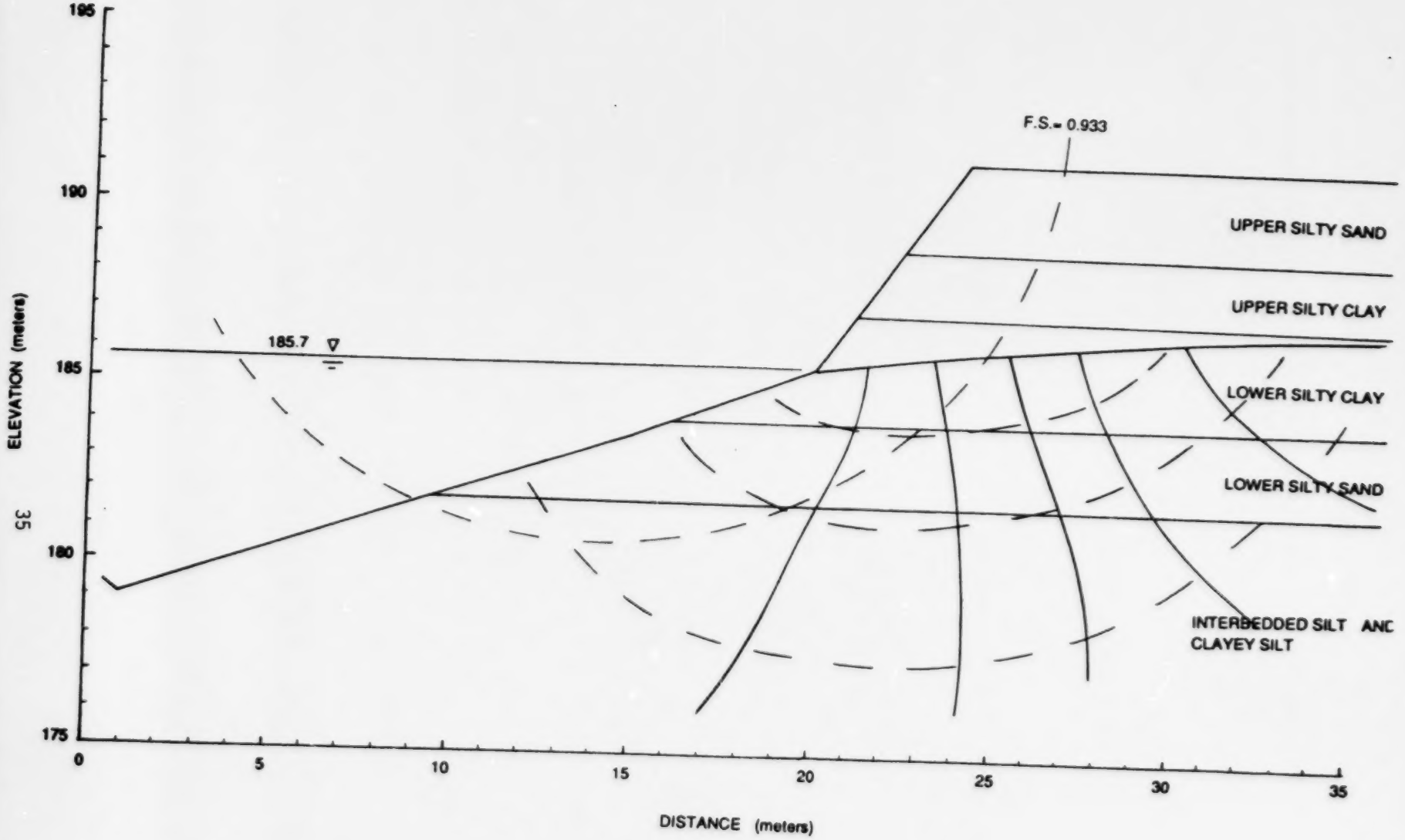


Figure 13 Slope stability analysis - Case 8.

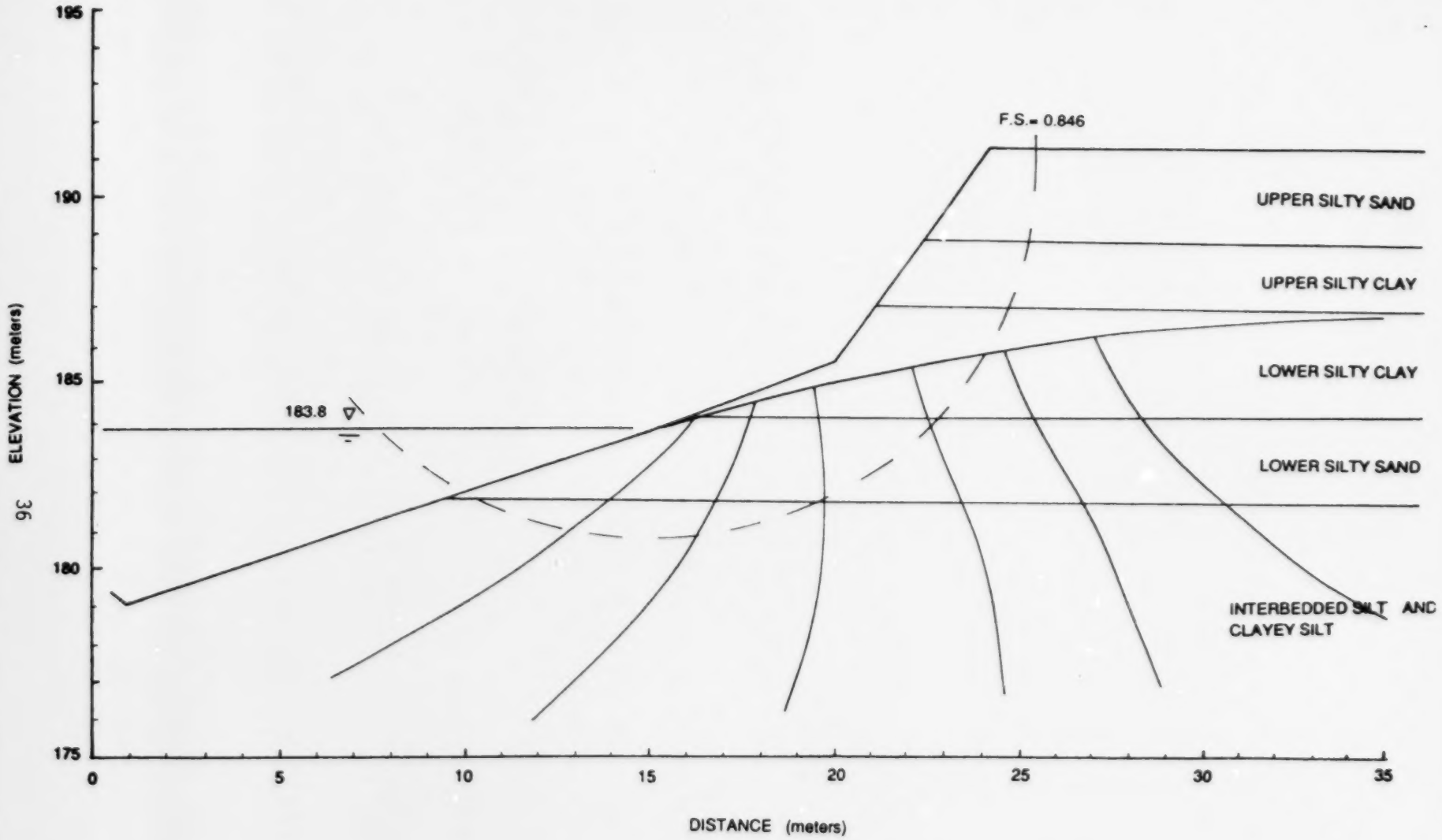


Figure 14 Slope stability analysis - Case 9.

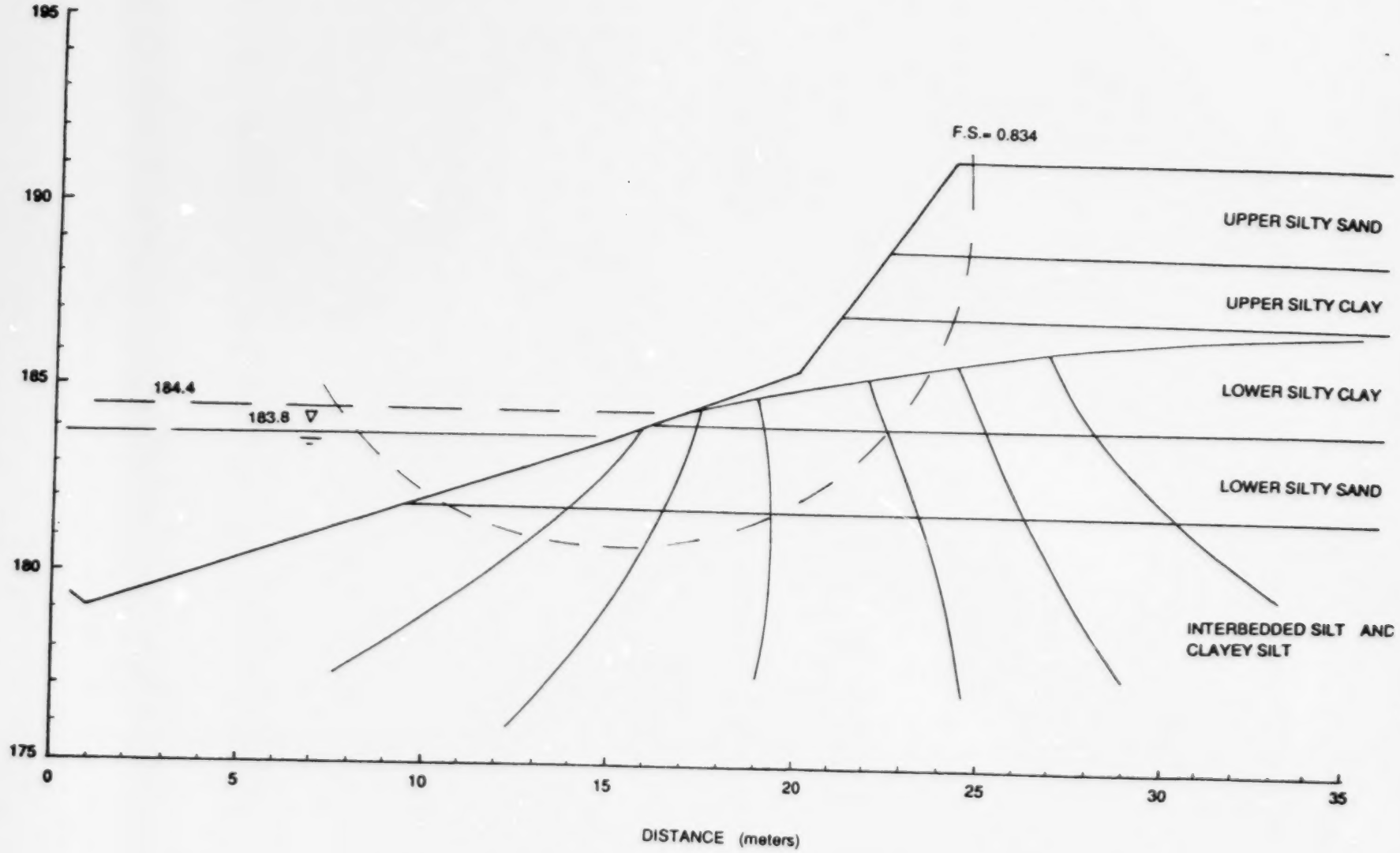


Figure 15 Slope stability analysis - Case 10.

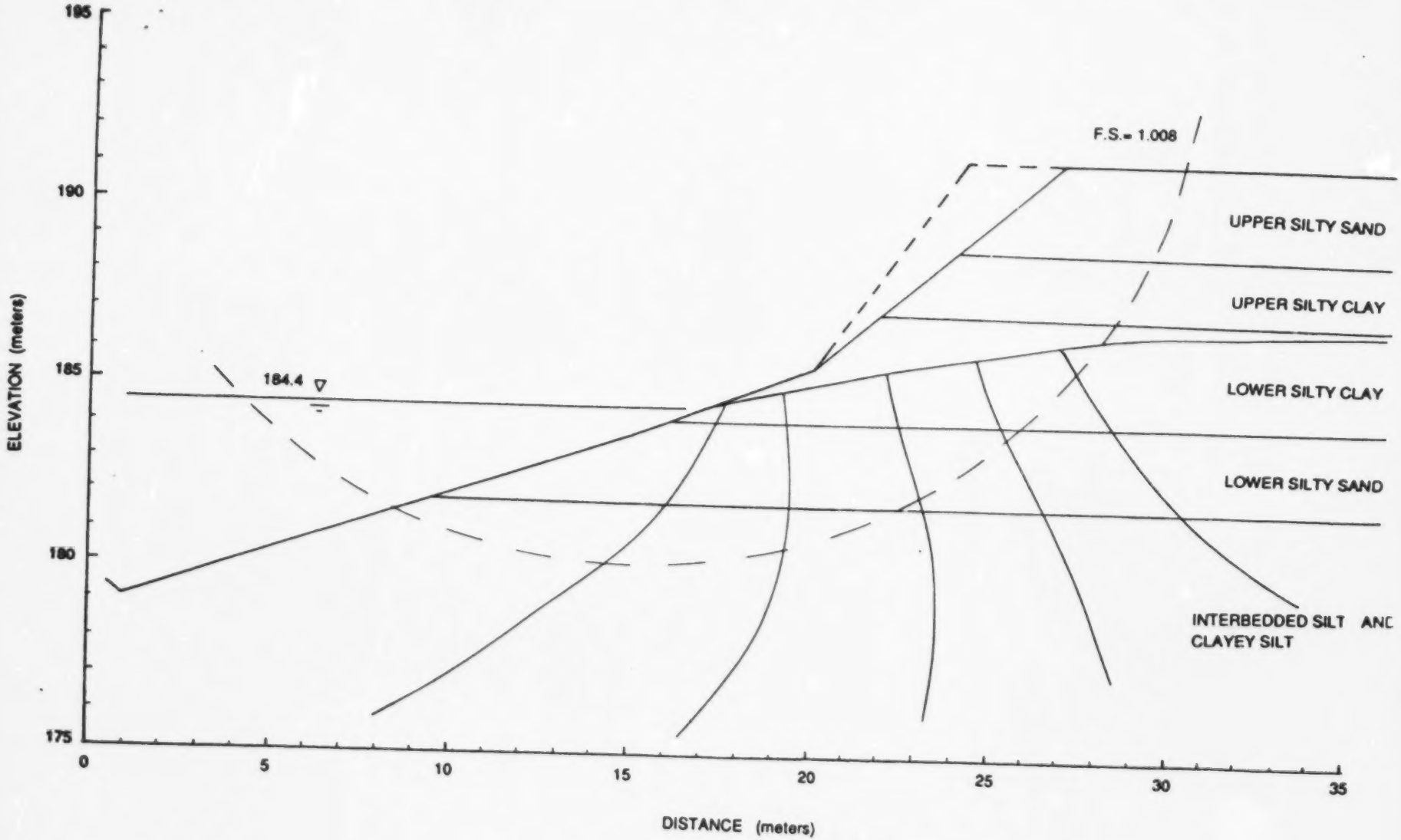


Figure 16 Slope stability analysis - Case 11.

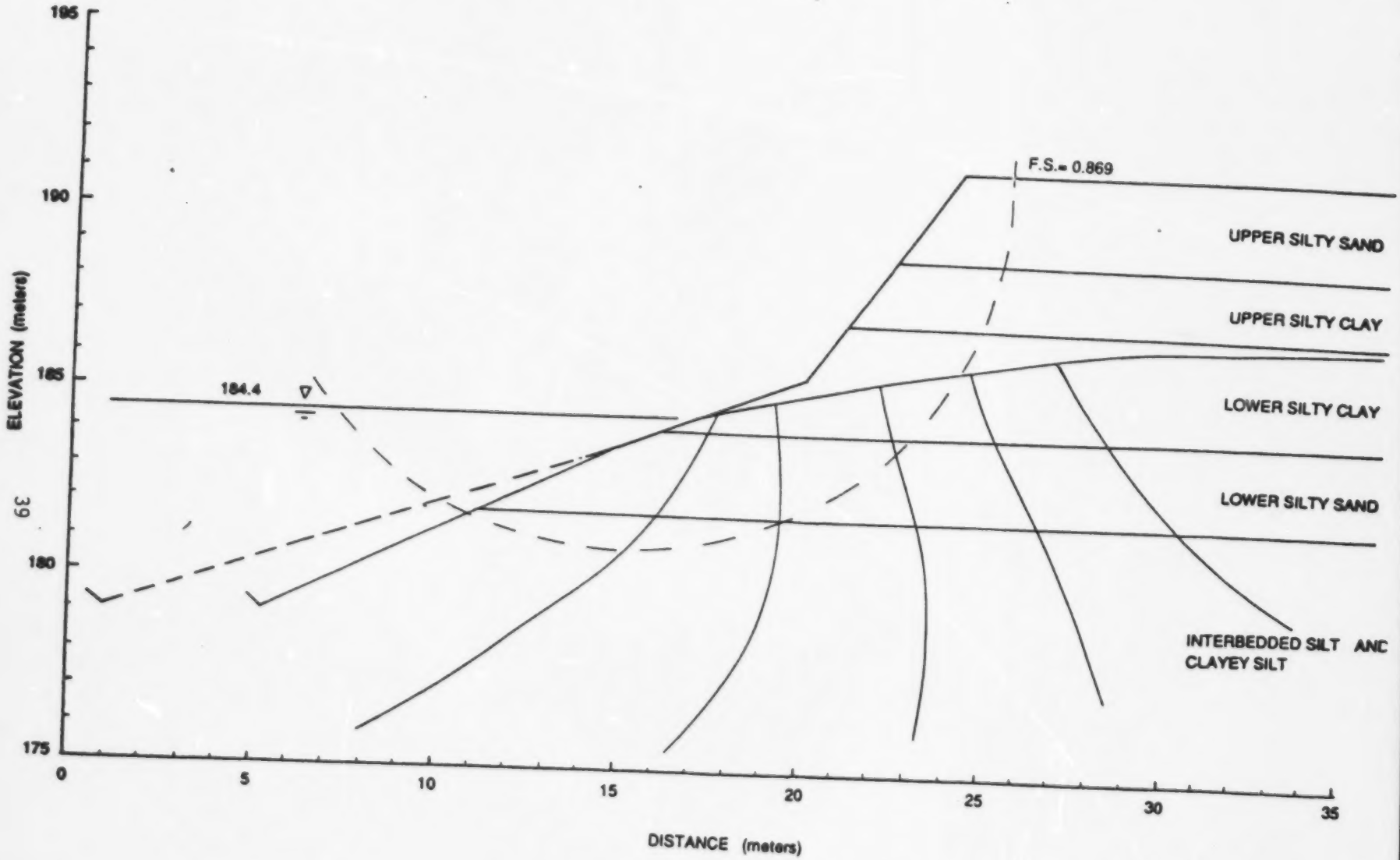


Figure 17 Slope stability analysis - Case 12.

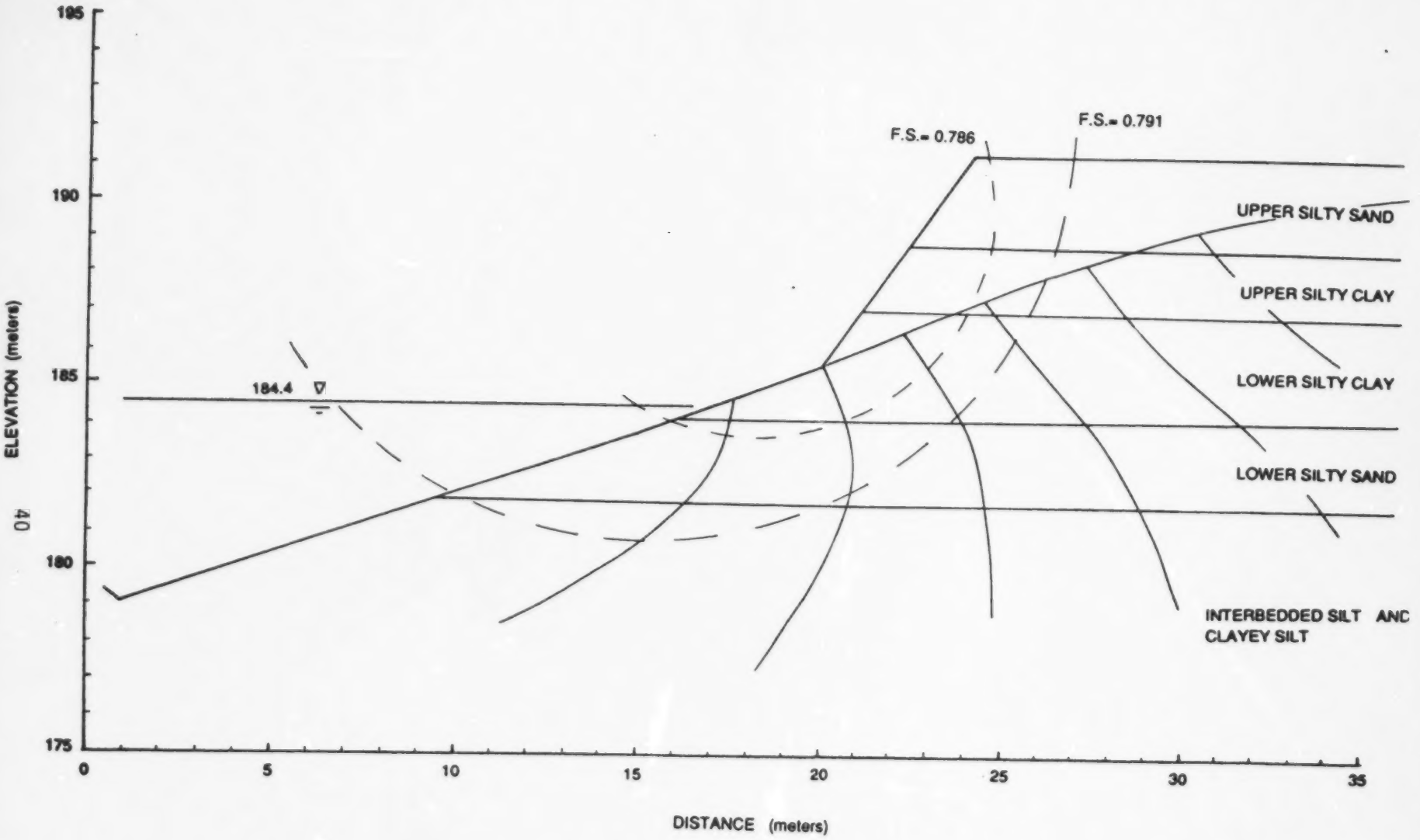


Figure 18 Slope stability analysis - Case 13.

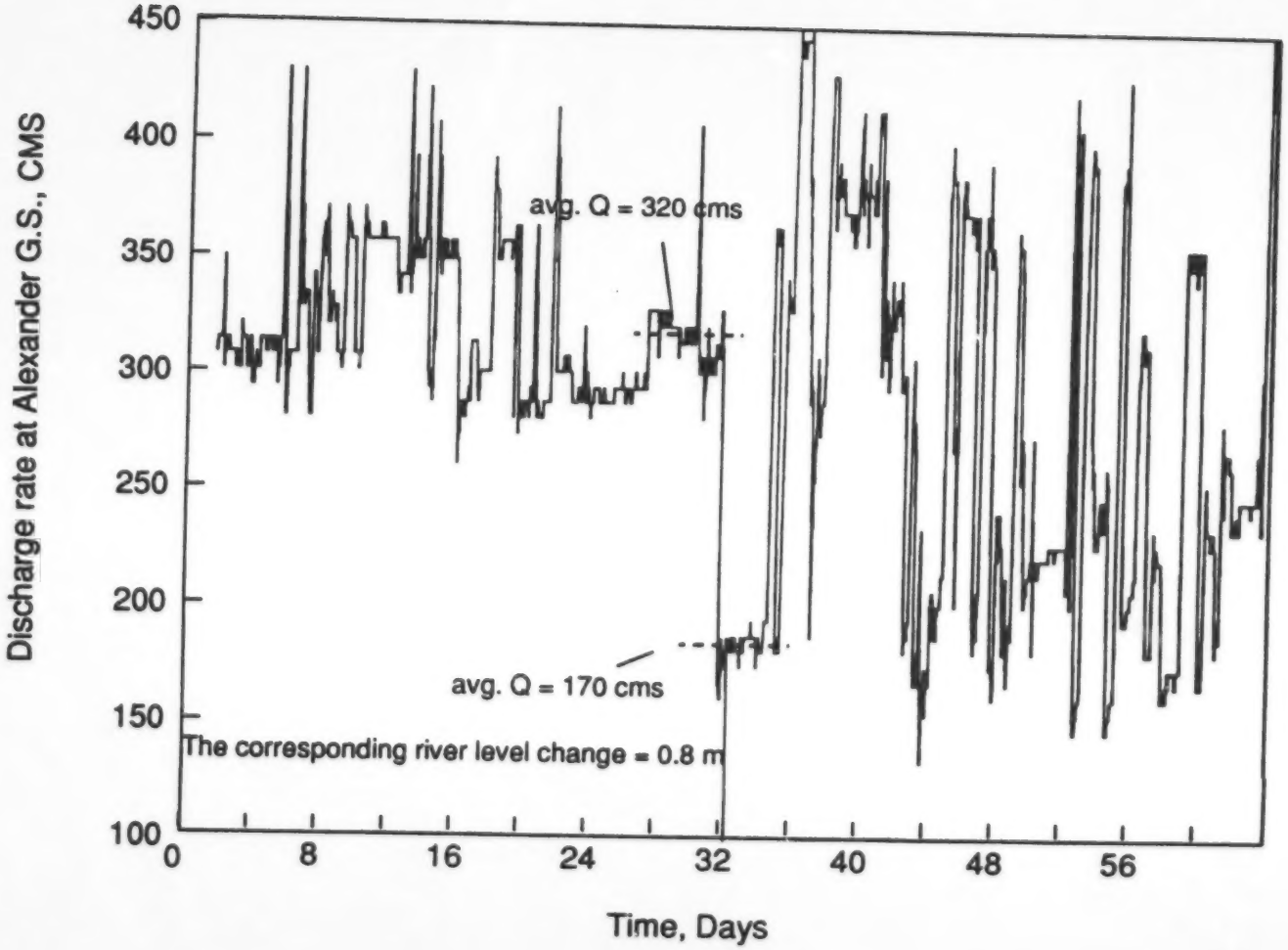


Figure 19 The hourly discharge at Alexander GS - April to May 1985.

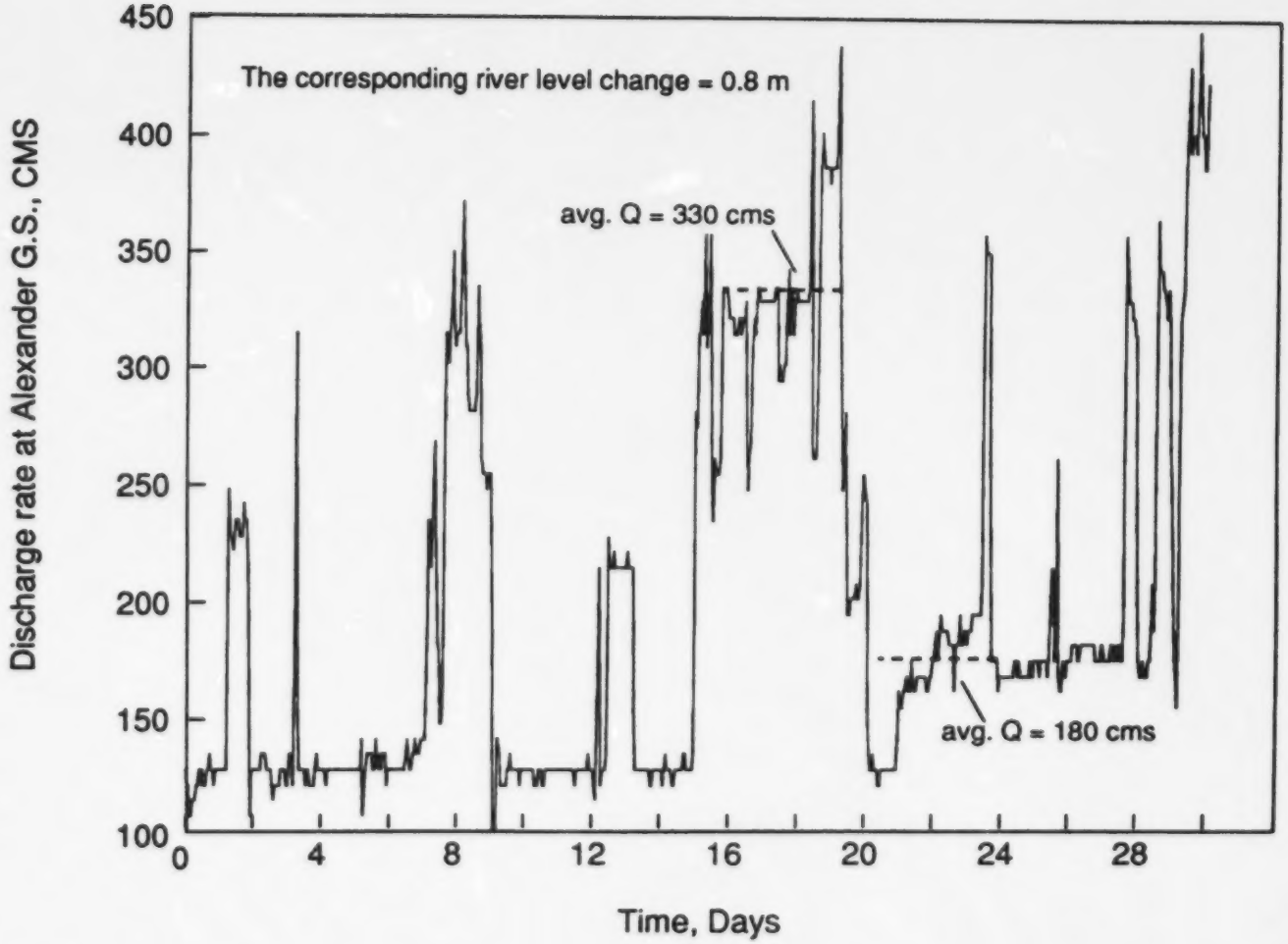


Figure 20 The hourly discharge at Alexander GS - May 1988.

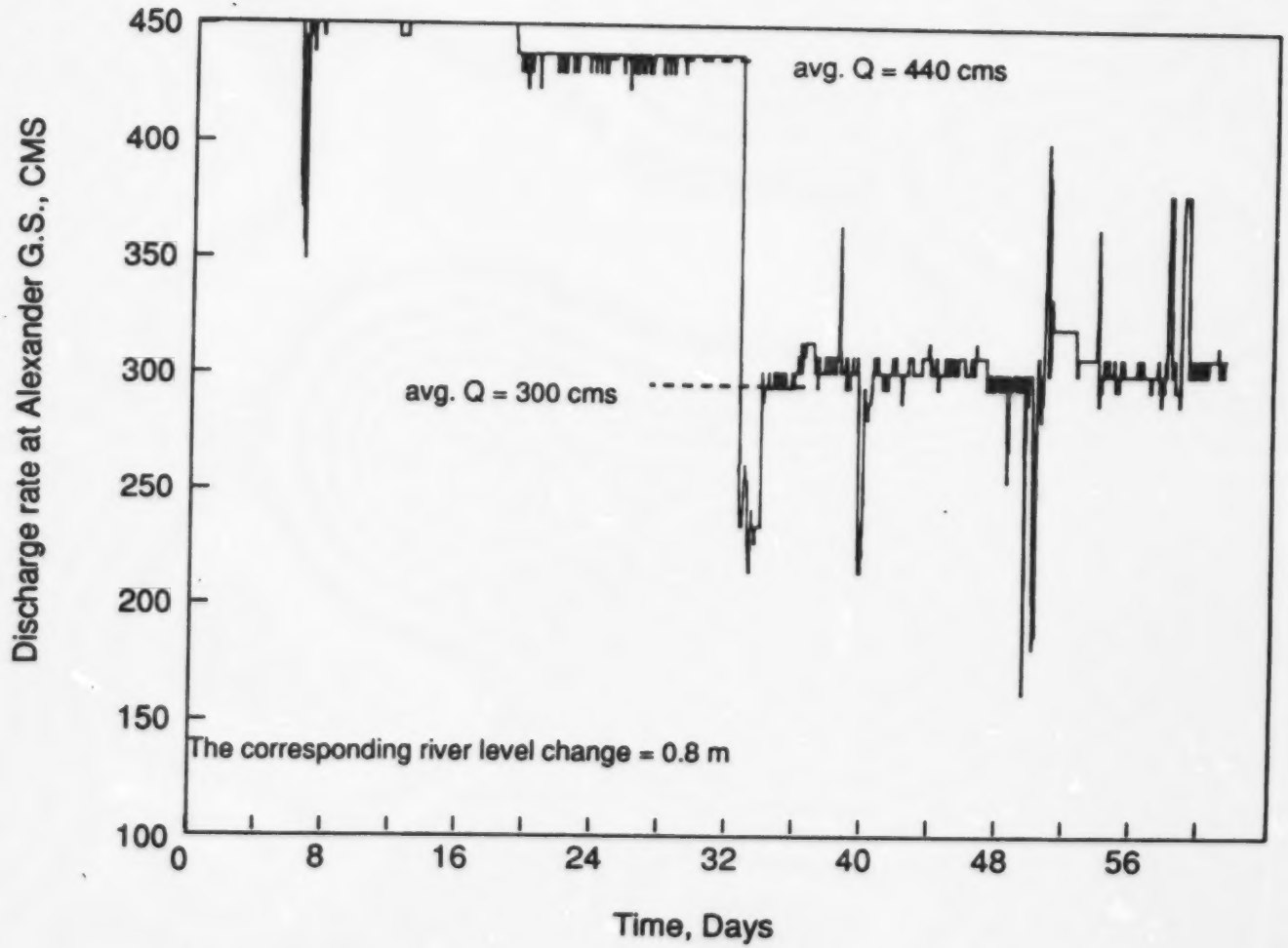


Figure 21 The hourly discharge at Alexander GS - Dec 1989 to Jan 1990.

APPENDIX A
Proposed Field Study 1991

The findings from Trow's geotechnical investigation and the parametric slope stability analysis carried out by us suggest that the presence of a soft lacustrine deposit, high groundwater conditions in the bank slopes and the river erosion led to the initial bank failure. But, the mechanism of retrogression of this failure to a large slide of April 1990 has not been identified due to a lack of information on strength properties of the weak silty clay layers and the effects of river level changes on the pore water pressure in the toe of the bank slope.

Bird and Hale Ltd. after an extensive study of the available airphotos of the area dating back to 1931 concluded that there is evidence of several landslides (retrogressive type in appearance) along the banks of the Nipigon River, most of which had similar soil conditions and erosional features as that of the 1990 failure area.

Considering the importance of both the land features and the effects of river erosion in the bank stability downstream from Alexander GS, a field checking of the soil conditions, bank geometry and the impact of river level fluctuations on the bank stability, is proposed to be carried out. Also, the conditions of the bank slope and errodability of soils in the river level fluctuation zone along Nipigon River should be studied.

The following field investigation is proposed to be carried out in February-March, 1991.

A. INVESTIGATION OF 1990 FAILURE SITE

1) Survey of the land in and around the failure area to reconstruct the bank geometry prior to failure.

2) Soil sampling in 3 boreholes (B1, B2 and B3) at locations shown in Figure B-1. Each borehole is put down to 15 m with continuous undisturbed sampling (piston samples) in the soft clay strata. A standpipe type piezometer will be installed in each of the boreholes to monitor groundwater level changes.

3) Electric piezocone tests will be performed at 3 locations, C1, C2 and C3, as shown in Figure B-1 to delineate the soil stratigraphy of the unfailed and failed areas.

B. INVESTIGATION OF POTENTIAL FAILURE SITE

Based on Bird and Hale's report, a bank slope with soil and groundwater conditions similar to those present at the 1990 failure site is selected for investigating the slope stability and impact of the river level fluctuations on the bank slope stability. The location of this site is shown in Figure B-1. The access to this bank slope for the drill rig and testing equipment is not yet assessed. If the access to this site is very difficult, an alternative site south of the 1990 failure area will be investigated (Figure B-2). Two boreholes, B4 and B5, and a piezocone test, C4, are proposed for this site.

In addition, an electrical piezometer (EP1) will be installed in the toe of the bank slope to monitor the pore water pressure response to the river level changes.

A laboratory testing program will be carried out to determine the shear strength parameters of the silty clay layer and the erodability of soils at the toe of the bank slopes.

A visual survey of the bank conditions (slopes and erosion features) along the Nipigon River downstream of Alexander GS is also proposed to be carried out in the summer or fall of 1991 by lowering the river level to its lowest level.

C. EVALUATION OF THE RIVER BANK SLOPES

An inspection of the bank slope condition will be made and the erosion damage to the slopes will be assessed as follows:

- visual inspection of river bank slopes
- mapping and photographing
- some land survey for bank heights and slopes
- collection of soil samples from the slopes in the zone of the river level fluctuation
- laboratory analysis for effects of wetting and drying and erodability of soil by river flow.

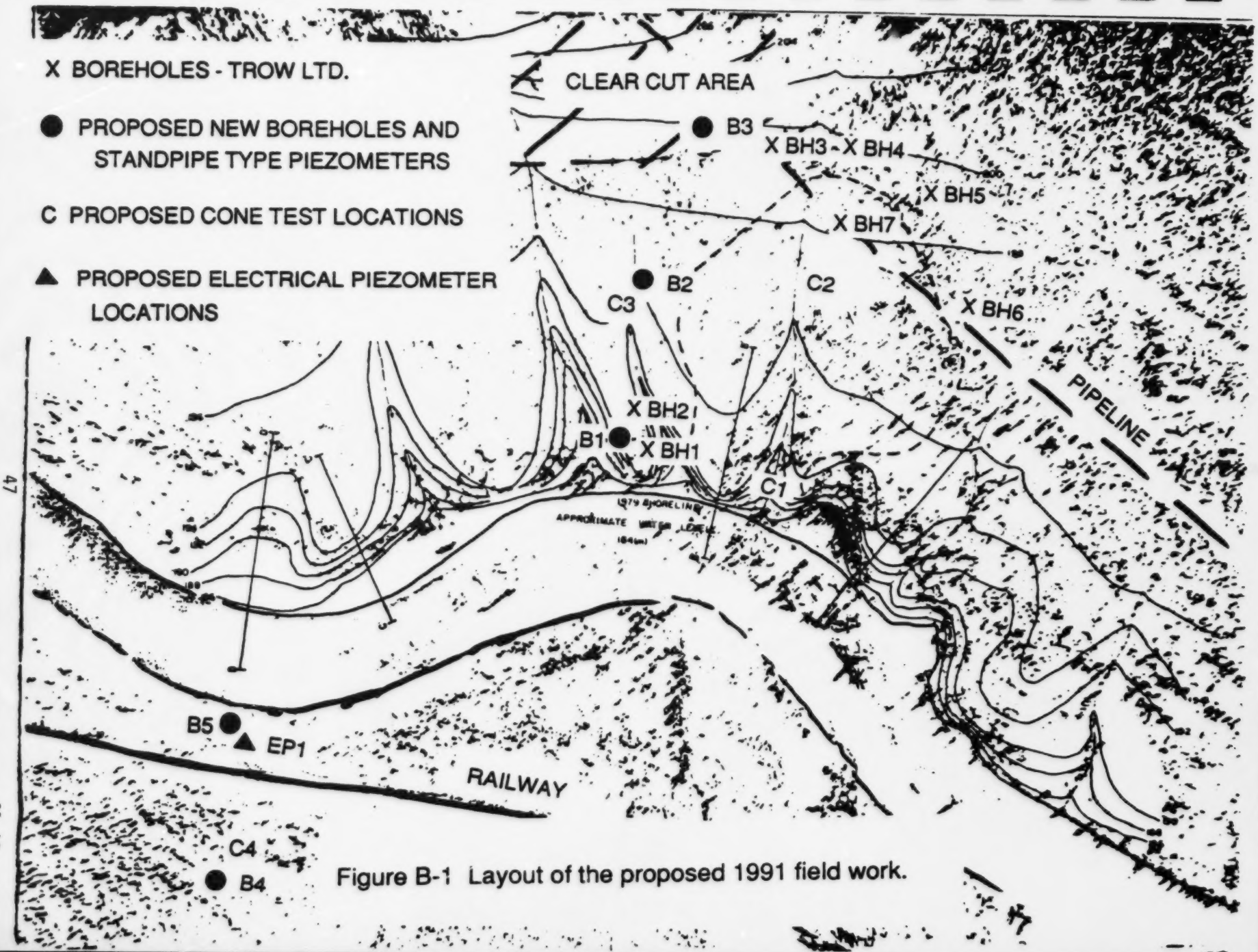


Figure B-1 Layout of the proposed 1991 field work.

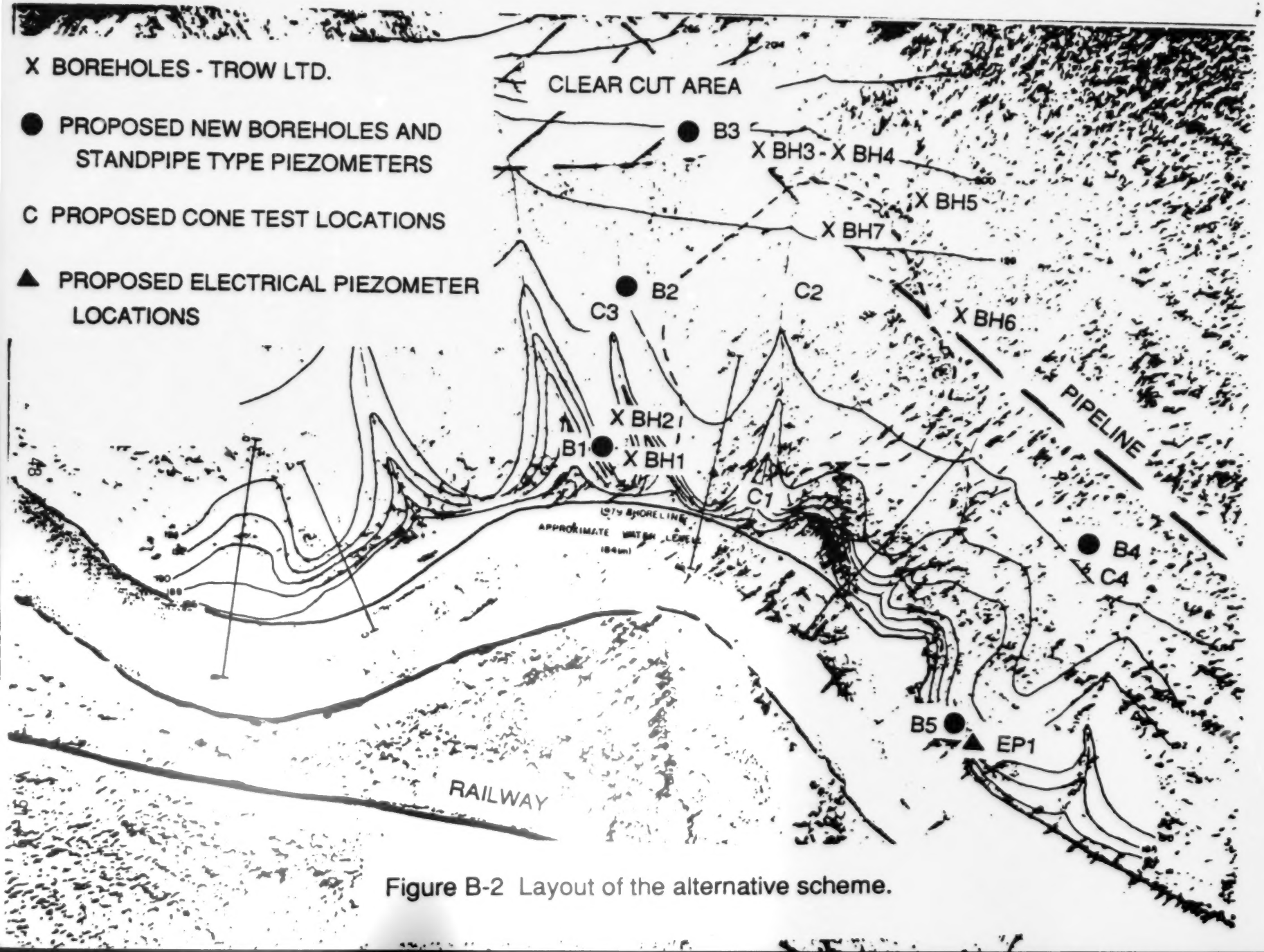


Figure B-2 Layout of the alternative scheme.